

CHESSMAN DAM REHABILITATION INVESTIGATION

PART I

**GEOTECHNICAL AND PRELIMINARY
ENGINEERING**

SUBMITTED TO:

**CITY OF HELENA
PUBLIC WORKS DEPARTMENT
316 NORTH PARK
HELENA, MONTANA 59623**

NOVEMBER 1984

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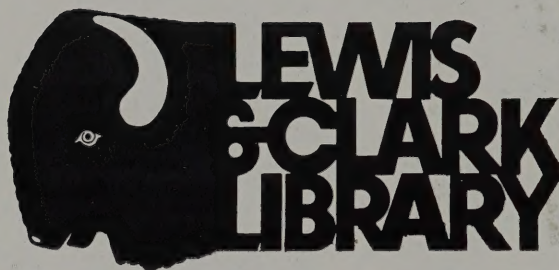
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PART I

CHESSMAN DAM REHABILITATION INVESTIGATION

1.0	EXECUTIVE SUMMARY	1-1
1.1	BACKGROUND	1-1
1.2	ENVIRONMENTAL STABILITY	1-2
1.3	SPILLWAY DESIGN FLOOD	1-3
1.4	OTHER STUDIES	1-4
1.5	CONCLUSIONS	1-5
1.6	RECOMMENDATIONS	1-7
2.0	PROJECT BACKGROUND	2-1
2.1	INTRODUCTION	2-1
2.1.1	Authority	2-1
2.1.2	Purpose of Study	2-1
2.2	DESCRIPTION OF PROJECT	2-2
2.2.1	General	2-2
2.2.2	Project Location	2-3
3.0	REHABILITATION INVESTIGATION	3-1
3.1	INTRODUCTION	3-1
3.2	PROJECT INFLOW DESIGN FLOOD	3-1
3.2.1	Classification and Applicable Guidelines	3-1
3.2.2	Probable Maximum Precipitation	3-4
3.2.3	Flood Characteristics	3-12
3.2.4	100 Year Design Flood Hydrographs	3-20
3.3	PROJECT HISTORY	3-21
3.3.1	Starting Reservoir Criteria	3-21
3.3.2	Department Freshwater	3-21
3.3.3	Existing Spillway Adequacy	3-22
3.3.4	Spillway Alternatives	3-23
3.3.5	Ranking of Spillway Alternatives	3-24
3.3.6	Selection of Spillway Alternative	3-27
3.4	Geotechnical Investigation	3-28
3.4.1	Field Investigation	3-30
3.4.2	Laboratory Investigation	3-34
3.4.3	Rock Description and Subsurface Profile	3-35
3.4.4	Stability Analysis Methods and Design Criteria	3-36
3.4.5	Results of Stability Analysis	3-42
3.4.6	Spillway Design for the Main Dam	3-44
3.5	PRELIMINARY ENGINEERING DESIGN AT SADDLE DAM	3-50
3.5.1	Topographic Survey	3-50
3.5.2	Forest Service Boundary (Corridor)	3-50

PROJECT NO. 500-107-01 (33)

Table of Contents (cont.)

Page

PART I

Chessman Dam Rehabilitation Investigation

Table of Contents

Page

1.0	EXECUTIVE SUMMARY	1-1
1.1	BACKGROUND	1-1
1.2	EMBANKMENT STABILITY	1-2
1.3	SPILLWAY DESIGN FLOOD	1-4
1.4	OTHER FINDINGS	1-6
1.5	CONCLUSIONS	1-6
1.6	RECOMMENDATIONS	1-7
2.0	PROJECT BACKGROUND	2-1
2.1	INTRODUCTION	2-1
2.1.1	Authority	2-1
2.1.2	Purpose of Scope	2-1
2.2	DESCRIPTION OF PROJECT	2-2
2.2.1	General	2-2
2.2.2	Description of Dam and Appurtenances	2-3
3.0	REHABILITATION INVESTIGATION	3-1
3.1	INTRODUCTION	3-1
3.2	PROJECT INFLOW DESIGN FLOOD	3-1
3.2.1	Classification and Applicable Guidelines	3-1
3.2.2	Probable Maximum Precipitation	3-4
3.2.3	Basin Characteristics	3-12
3.2.4	Inflow Design Flood Hydrographs	3-20
3.3	PROJECT ROUTING	3-21
3.3.1	Starting Reservoir Criteria	3-21
3.3.2	Embankment Freeboard	3-21
3.3.3	Existing Spillway Adequacy	3-22
3.3.4	Spillway Alternatives	3-23
3.3.5	Routing of Spillway Alternatives	3-24
3.3.6	Selection of Spillway Alternative	3-27
3.4	GEOTECHNICAL EVALUATION	3-28
3.4.1	Field Investigations	3-30
3.4.2	Laboratory Investigations	3-34
3.4.3	Area Description, Geology and Subsurface Profile	3-35
3.4.4	Stability Analysis - Methods and Design Criteria	3-39
3.4.5	Results of Stability Analysis	3-42
3.4.6	Rehabilitation Alternatives for the Main Dam Embankment	3-44
3.5	PRELIMINARY ENGINEERING DESIGN AT SADDLE DAM	3-50
3.5.1	Topographic Survey	3-50
3.5.2	Forest Service Roadway (Causeway)	3-50

Table of Contents (cont.)

	Page
3.0 <u>REHABILITATION INVESTIGATION (cont.)</u>	
3.5.3 Hydraulic Considerations	3-51
3.5.4 Structural Considerations	3-54
3.5.5 Geotechnical Considerations	3-57
3.5.6 Preliminary Design Drawings	3-58
3.5.7 Cost Estimate	3-58
3.5.8 Recommendation	3-58
3.6 EMERGENCY WARNING PLAN (UNDER SEPERATE COVER PART II)	
4.0 <u>RECOMMENDATIONS FOR PROJECT REHABILITATION</u>	
4.1 <u>PROJECT OPERATION</u>	4-1
4.2 MAIN DAM	4-1
4.3 SADDLE DAM	4-4
4.4 COST ESTIMATES AND SCHEDULING	4-7
4.5 SCHEDULING	4-12
<u>LIST OF FIGURES</u>	
1. Location Map	1-8
2. Analytical Procedures for Sizing Dam Spillways	3-5
3. PMP Storm A	3-8
4. PMP Storm B	3-10
5. PMP Storm C	3-11
6. Main Channel Profile - Chessman Dam Drainage	3-14
7. Chessman Reservoir - Drainage Basin Map	3-15
8. Soil Conservation Service Unit Hydrographs	3-19
9. Proposed Spillway Ratings	3-25
10. Spillway Design Flood	3-26
11. Location of Drill Holes	3-33
12. Typical Geologic Section Along Dam Axis	3-38
13. Topographic Survey at Saddle Dam	3-51
14. Spillway Structure Alternate 1	3-54
15. Spillway Structure Alternate 2	3-561
16. Plan View Spillway Structure	3-59
17. Profile Spillway Structure	3-60
18. Plan View Main Dam Rehabilitation	4-5
19. Cross Section Main Dam Rehabilitation	4-6
20. Construction Schedule and Project Activity Diagram	4-13
<u>LIST OF TABLES</u>	
Pertinent Data (revised April 1984)	1-9
1. PMP and Snowmelt Totals - PMP Storm A	3-7
2. Soil Conservation Service Unit Hydrograph Procedures	3-18
3. Construction Cost Estimate - Spillway Structure Alternate 1 - Riprap	3-61
4. Construction Cost Estimate - Spillway Structure Alternate 2 - Concrete Drop	3-62
5. Preliminary Cost Estimate - Chessman Dam Rehabilitation	4-8

APPENDIX

Portions of Northern Engineering and Testings Geotechnical Reports

City of Helena's Piezometer Readings

Dr. H. Bolton Seed's Letter Report

City of Helena's TV log - Outlet Conduit

1980, the City of Helena, Montana, and Northern Engineering and Testing, Inc., conducted a rehabilitation investigation of the Chusman Dam. The investigation was to address deficiencies noted in the June 1980, Phase I Dam Safety Report - "Chusman Main Dam and Chusman Saddle Dam", and develop a rehabilitation plan for the project. The analysis was to address geotechnical, hydrologic, hydraulic and public safety requirements necessary for the project to operate at full capacity. To accomplish this it was anticipated that assessment of stability of the Main Dam and spillway capacity of the Saddle Dam were the primary areas of rehabilitation concern and would require preliminary engineering design and cost estimates to adequately address the level of future repair efforts.

The results of this investigation are presented in two parts. The preliminary engineering and cost estimates for rehabilitation are included as Part I. Part II, under separate cover, includes the Emergency Warning Plan which sets forth identification, operation and notification procedures for an emergency situation.

1.1 BACKGROUND

The present Chusman Reservoir Project was completed in 1964 as part of the City of Helena's Tamele Water System and augments supplies from the Tamele Creek drainage. The facility is owned and operated by the City of Helena under a Forest Service special use permit. The project includes a Main Dam and a Saddle Dam with a spillway.

In 1980, an inspection of Chusman Main Dam and Chusman Saddle Dam was conducted under the National Dam Safety Program by Harrison-Maleris, Inc. This inspection revealed several deficiencies and recommended that further investigation be conducted to determine a spillway design fixed to assess the stability of the Main Dam and to prepare a downstream warning plan.

APPENDIX
Portions of Northern Engineering and Testing Technical Reports
City of Helena's Firewater Reservoir
Dr. H. Holten's Test Report
City of Helena's TV Log - Boiler Control

1.0 EXECUTIVE SUMMARY

In accordance with a contract with the City of Helena, dated June 6, 1983 Morrison-Maierle, Inc., and Northern Engineering and Testing, Inc., conducted a rehabilitation investigation of the Chessman Dams. The investigation was to address deficiencies noted in the June 1980, Phase 1 Dam Safety Report - "Chessman Main Dam and Chessman Saddle Dam", and develop a rehabilitation plan for the project. The analysis was to address geotechnical, hydrologic, hydraulic and public safety requirements necessary for the project to operate at full capacity. To accomplish this it was anticipated that embankment stability of the Main Dam and spillway capacity of the Saddle Dam were the primary areas of rehabilitation concern and would require preliminary engineering designs and cost estimates to adequately address the level of future repair efforts.

The results of this investigation are presented in two parts. The preliminary engineering and cost estimates for rehabilitation are included as Part I. Part II, under separate cover, includes the Emergency Warning Plan which sets forth identification, operation and notification procedures for an emergency situation.

1.1 BACKGROUND

The present Chessman Reservoir Project was completed in 1908 as part of the City of Helena's Tenmile Water System and augments supplies from the Tenmile Creek drainage. The facility is owned and operated by the City of Helena under a Forest Service special use permit. The project includes a Main Dam and a Saddle Dam with a spillway.

In 1980, an inspection of Chessman Main Dam and Chessman Saddle Dam was conducted under the National Dam Safety Program by Morrison-Maierle, Inc. This inspection revealed several deficiencies and recommended that further investigation be conducted to determine a spillway design flood to assess the stability of the Main Dam and to prepare a downstream warning plan.

The first part of the paper discusses the importance of the research and the objectives of the study. It also outlines the methodology used in the study and the data sources. The second part of the paper presents the results of the study and discusses the implications of the findings. The third part of the paper concludes the study and provides recommendations for future research.

The results of the study show that there is a significant positive relationship between the variables studied. This finding is consistent with the previous research in the field. The implications of the findings suggest that the variables studied are important factors in the process being studied. The recommendations for future research are based on the findings of the study and aim to address the limitations of the current study.

The study has several limitations, including the sample size and the scope of the study. Despite these limitations, the study provides valuable insights into the process being studied. The findings of the study are based on the data collected and the analysis performed. The conclusions drawn from the study are based on the evidence presented.

The study is a contribution to the knowledge in the field and provides a basis for further research. The findings of the study are presented in a clear and concise manner, making it easy to understand. The study is a valuable resource for researchers and practitioners in the field.

1.2 EMBANKMENT STABILITY

The 1980 Dam Safety Inspection Report established that the Main Dam revealed evidence of seepage in the embankment which could affect the stability of the dam and the safety of the project. For that reason the primary emphasis of the geotechnical investigation performed under this contract was placed on determining actual field conditions and verifying the stability of the Main Dam.

The geotechnical evaluation consisted of three phases: 1) field investigation (drill holes, test pits, piezometers etc); 2) laboratory tests and field monitoring; and 3) stability analysis. Samples were taken at various levels and locations in the dam embankment and in the undisturbed foundation to characterize the site conditions for the technical analysis. Piezometers were installed to measure the levels of seepage flows in the embankment compared to the reservoir level. The piezometers were monitored over the entire period of study at varying reservoir levels. The dam's concrete core wall was exposed and core samples taken to assess its strength and integrity. Samples were tested to determine their structural properties as related to design and stability parameters.

An additional consideration for stability is the potential for seismic activity in the region. The project is located in a seismic zone 3. Record of seismic activity in the area dates to 1869 and the dam was in place during the 1935 earthquake which registered magnitudes of 6.0 and 6.3 on the Richter Scale and was centered north of Helena about 12 miles from the site. That earthquake was one of the largest registered in Montana and there is no record found which identifies damage to the dam. In spite of the fact the dam seemingly survived that earthquake there is sufficient evidence to indicate that another quake of similar magnitude could occur in the vicinity of the project approximately every 70 years. This concern dictates stringent design standards for the rehabilitation of the project.

Findings:

1. The concrete core wall in the Main Dam is effectively intact and functions as a cutoff to seepage flow. It is not, however, founded on bedrock or impervious material throughout its length and an old streambed creates a major seepage path beneath the corewall.

2. The stability analysis of the existing main embankment with a ground operating reservoir and under various design conditions showed the structure not to meet recommended factors of safety. An actual safety factor less than one indicates the structure will fail if the conditions used in the analysis occur. A comparison of actual to standard safety factors is as follows:

<u>Design Condition</u>	<u>Actual Safety Factors</u>	<u>Standard Safety Factors</u>
Static (without earthquake)	1.1	1.5
Seismic	0.7	1.0
Rapid Reservoir Drawdown	1.0	1.2

The seismic safety factor indicates a critical condition exists, and that the structure will fail if the standard design earthquake occurs.

3. The embankment drilling and sampling program identified zones or lenses of unconsolidated sands within the embankment and in the undisturbed foundation material. In the foundation where the sands are also continuously saturated, the condition presents the potential for sudden embankment failure during an earthquake due to the phenomenon of liquefaction. During a significant seismic event the saturated sands could actually liquify causing massive slipping and settling of the embankment. This condition and analysis represents a totally different design concern and problem from the seismic analysis identified previously. It does not lend itself to the computation of a factor of safety or to a prediction of probability of failure.

These critical sand zones were encountered unexpectedly during the field exploration program and required special analysis to assess the extent and magnitude of the problem. A reorientation of the original scope of services was authorized to allow for an immediate expansion of the field exploration program to obtain additional site data. The data and analysis were reviewed with Dr. H. Bolton Seed, a recognized expert in the field of earthquake engineering currently on staff at the University of California at Berkeley. Dr. Seed concurred with the analysis of the geotechnical engineers. He agreed that the loose sand layers may liquify under the earthquake ground shaking which has the potential of occurring in the vicinity of the dam. Dr. Seed also expressed the unpredictability of the extent of failure or damage to the embankment should liquefaction occur.

1.3 SPILLWAY DESIGN FLOOD

Because of its size and hazard classification, the Chessman Project must be able to pass the probable maximum flood (PMF). An important consideration in the adequacy of the project is the determination of the PMF that the dam and spillway will be subjected to. Probable maximum precipitation (PMP) amounts producing the PMF were developed from a recent report developed from the National Weather Service and an independent analysis by a Bureau of Reclamation meteorologist. These sources of data were not available for the Phase I Inspection Report.

Three storms are considered in this evaluation for the Chessman area. Storm A indicates an accumulated runoff of 18.4 inches in a period of 24 hours and 21.9 inches in 72 hours. Storm B projects a total of 22.9 inches of runoff in 24 hours and 26.9 inches in 72 hours. Storm C showed a total 11.5 inches of runoff in 6 hours.

May and June are considered to be the most likely periods for extreme rainfall event to combine with snowmelt and/or extreme infiltration conditions to produce a PMF. Storm A could occur in May and Storm B in June. A routing analysis determined that Storm B would produce the worst case for spillway adequacy. Peak inflow during design Storm B would equal 5,400 cfs. Routing of this storm with the existing spillway would allow the

reservoir to rise to elevation 104.1 feet which would overtop the Saddle dam embankment and the corewall in the Main Dam. Therefore, the existing spillway is inadequate to route the PMF and alternates which consider Storm B were developed.

Two spillway operating alternatives were considered, one with a crest elevation of 97 feet (project datum) and other with a crest at 99 feet. The lower crest elevation would require eliminating 200 acre-feet or 65 million gallons from the storage capacity of the reservoir. Water needs and added benefits with spillway crest at elevation 99 feet along with a slight increase in cost lead to a recommendation of this operating alternate. Increases above the storage level of 99 feet were considered but costs, inflow limitations and water rights indicate the potential for development of additional water at the site to be marginal.

Two spillway structural alternatives were considered; riprap protection of the existing Saddle Dam and a concrete drop spillway. The riprap protection of the existing dam is recommended based on anticipated infrequent useage and a cost comparison of the structural alternatives.

Routing of design Storm B with the proposed riprap protected spillway structura will produce a maximum outflow equal to 2,100 cfs. The reduction from 5,400 cfs inflow, to 2,100 cfs outflow occurs due to the significant storage in the reservoir.

To evaluate downstream conditions and available response times in the remote event of a sudden failure of the Main Dam a dambreach analysis was run. The analysis provided the basis for an emergency warning plan which included identification, emergency operation and repair and notification procedures. Travel times of the dambreach flood were computed as 25 minutes at Rimini, 2 hours at the Tenmile Settling Ponds and 3½ hours at State Nursery, as referenced from the time of dam failure. Instructions for emergency situations at Chessman are found under separate cover entitled Chessman Rehabilitation Investigations Part II - Emergency Warning Plan - June 1984.

1.4 OTHER FINDINGS

During the course of this analysis the future of the City of Helena's water supply was being evaluated by an independent consultant. Their evaluation process included costs for all components of the Tenmile Water Supply System including the Chessman rehabilitation and compared them with other water supplies and treatment systems. The costs estimate for restoring the Chessman Project to current engineering design standards were at an early stage of the rehabilitation investigation projected at one million dollars.

A television inspection of the existing outlet works by City staff on August 16, 1984 indicated that the condition of the cast iron pipe is adequate, while the concrete conduit requires only minimal repairs after 80 years of use. This discovery reduced earlier rehabilitation cost estimates by approximately \$200,000.

Because of the potential instability of the Main Dam embankment due to both static and dynamic forces, the City chose to drain the Chessman reservoir during the summer of 1984. Conclusions of this rehabilitation investigation and the City's overall water supply/treatment evaluation will enable decisions to be made as to the future operation of the project.

1.5 CONCLUSIONS

The analysis of alternate rehabilitation plans available to the City has addressed a range of options from various embankment and spillway modifications, to a totally new dam. The analysis has been performed in accordance with design standards and guidelines accepted by federal agencies. Some of the design criteria reflect ultimate or conservative conditions as is the case with hydrologic or seismic conditions. It is possible that use of lower design standards would allow the City to rehabilitate the structure for less cost by accepting a certain amount of increased risk. A decision to design to lower standards would need to be made in cooperation and agreement with the U.S. Forest Service and the State of Montana.

The costs of the alternatives recommended and presented in this report are based on site data and information and are sufficiently detailed to assess project feasibility and should be used in that context. If a reduction of 25 percent in project cost would effect the decisions to be made with regard to project feasibility, then further consideration should be given to conducting more detailed site evaluation and an analysis of risk and design criteria.

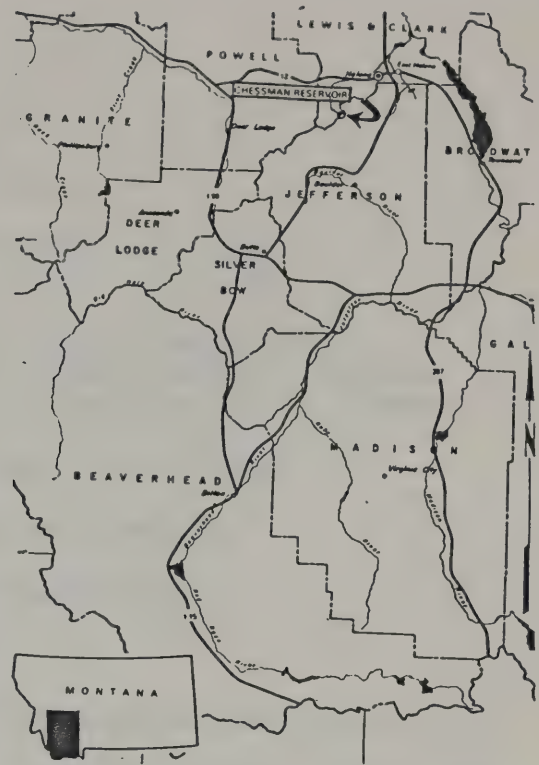
Presently the project can be rehabilitated at a cost of \$825,000 which will provide 531 million gallons of storage to the Tenmile Water System. This is a cost of \$0.0016 per gallon of raw water stored.

1.6 RECOMMENDATIONS

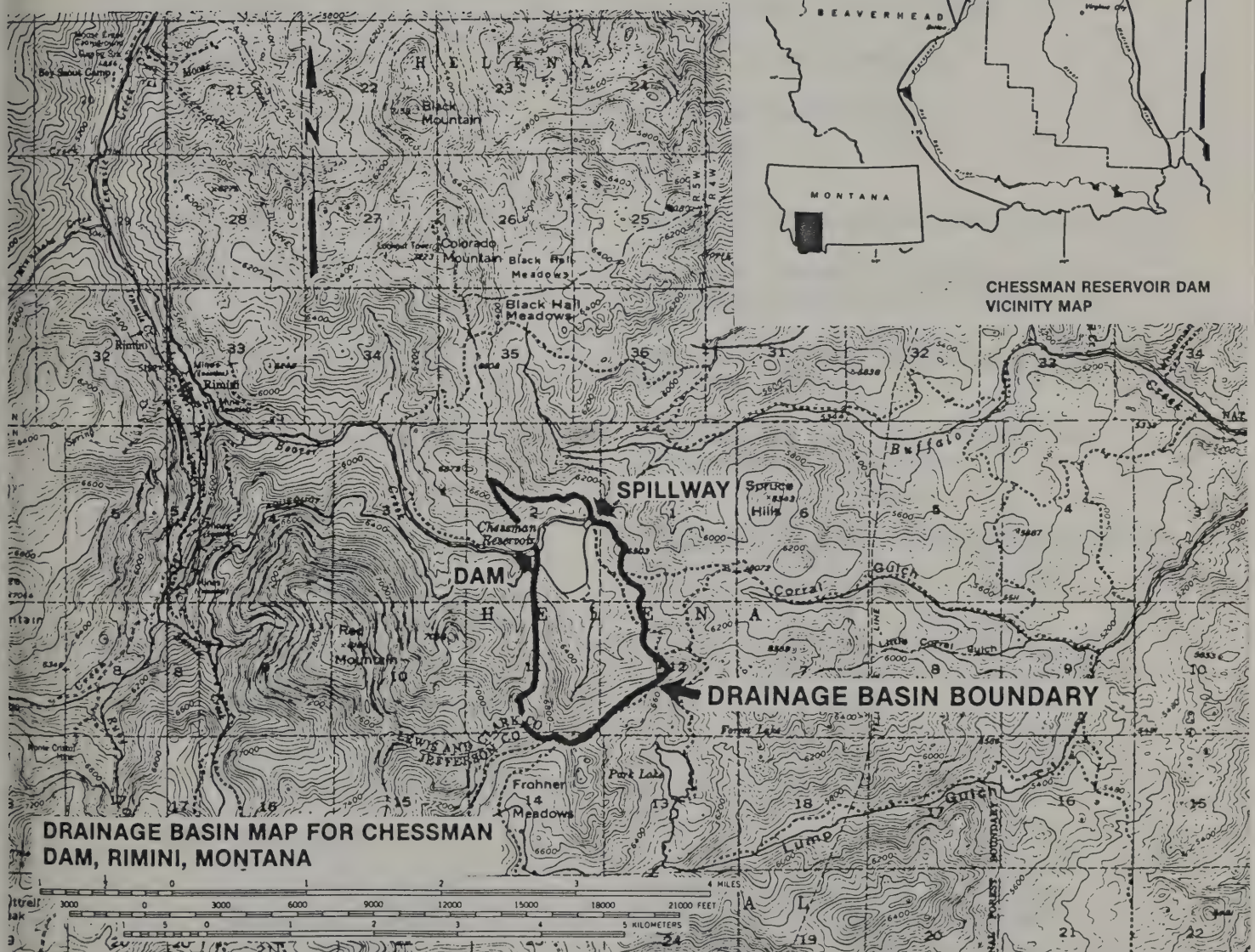
To restore the project to operational status and be in compliance with current engineering guidelines it is proposed that the following items be considered:

- * flattening the upstream slope of the Main Dam to $2\frac{1}{2}H$ to $1V$,
- * widening the crest width from 15 to 45 feet,
- * flattening the downstream slope to $3H$ to $1V$,
- * installing pressure relief wells in the foundation soils at the toe of the dam,
- * compaction grouting of loose sands in the foundation and embankment downstream of the cutoff wall,
- * extending the concrete cutoff wall up to the top of the dam,
- * reconstructing the spillway in the Saddle Dam and
- * extending the outlet works both upstream and downstream

The estimated cost of these improvements is \$825,000.



CHESMAN RESERVOIR DAM
VICINITY MAP



DRAINAGE BASIN MAP FOR CHESMAN
DAM, RIMINI, MONTANA

CHESMAN DAM REHABILITATION INVESTIGATION LOCATION MAP

FIGURE 1



PERTINENT DATA
(revised April 1984)

CHESSMAN DAM, MT-1090; CHESSMAN SADDLE DAM, MT-1562

LOCATION: Township 8 North, Range 5 West
Section 2 latitude 112°11', longitude
46°28' Lewis and Clark County, Montana

WATERSHEDS: Beaver Creek, Buffalo Creek

OWNER/OPERATOR: City of Helena, Montana

PURPOSE: Municipal Water Supply

YEAR CONSTRUCTED: 1908

MAIN DAM: Earthfill dam with concrete core wall

SADDLE DAM: Earthfill dam with concrete core wall
Project spillway located in the dam

MAIN DAM CLASSIFICATION: Intermediate size, high downstream
hazard potential

SADDLE DAM CLASSIFICATION: Intermediate size, significant down-
stream hazard potential

KEY ELEVATIONS:

	(Project Datum)	(Revised City Datum)
Top of main dam	Elevation 105 feet	34 feet
Top of main dam core wall	101 feet	30 feet
Top of saddle dam and core wall	102 feet	31 feet
Top of causeway road berm	101 feet	30 feet
Maximum operating pool	100 feet	29 feet
Normal operating pool	99 feet	28 feet
Spillway Crest	97 feet	26 feet
Outlet at toe of main dam	65 feet	-- feet

MAIN DAM DIMENSIONS:

Hydraulic Height	40 feet from outlet
Structural Height	57 feet from bottom of core wall
Crest Length	440 feet
Crest Breadth	15-17 feet
Face Slopes	1V on 2H

PERTINENT DATA (Continued)

SADDLE DAM DIMENSIONS:

Hydraulic Height	17 feet from toe of chute
Structural Height	27 feet from bottom of core wall
Total Length	105 feet
Crest Breadth	10 feet, concrete paved
Face Slopes	Upstream 1V on 1.5H Downstream 1V on 2.5H

SPILLWAY: (Saddle Dam)

Location	Left center of saddle dam
Type	Concrete-lined chute with broad crested weir control
Width	22.5 feet
Control	Provisions for flashboards to elevation 102 feet. Causeway road berm upstream at elev. 101 feet blocks spillway approach.
Crest Elevation	97 feet
Chute Slope	1V on 3H
Capacity at Overtopping	590 cfs, elevation 102 feet

OUTLET WORKS: (Main Dam)

Type	Submerged 16-inch-diameter Cast Iron (CI) inlet pipe discharging to a 24-inch by 27-inch concrete arch conduit.
Length	100 feet CI pipe 88 feet concrete arch conduit
Control	Gate valve located in dry-well valve chamber at center of dam.
Capacity	Approximately 25 to 30 cfs at normal pool.

RESERVOIR:

Surface Area	114 acres @ Normal Pool
Storage at:	
- Top of Main Dam	2370 acre-feet @ elev. 105 feet
Top of Saddle Dam	1990 acre-feet @ elev. 102 feet
Normal Pool	1630 acre-feet @ elev. 99 feet
Spillway Crest	1430 acre-feet @ elev. 97 feet

CHESS3/E

2.0 PROJECT BACKGROUND

2.1 INTRODUCTION

2.1.1 Authority

The rehabilitation investigation of the Chessman Project was conducted in accordance with a contract for services between the City of Helena and Morrison-Maierle, Inc. Consulting Engineers, dated June 6, 1983 and revised on January 19, 1984. Geotechnical services were provided by Northern Engineering and Testing, Inc. of Great Falls, Montana under a sub-contract with Morrison-Maierle, Inc.

2.1.2 Purpose and Scope

The Chessman Reservoir Project is owned and operated by the City of Helena as a municipal water supply facility under a U.S. Forest Service special use permit. The project is part of the city's Tenmile Water Supply System and augments supplies from Tenmile Creek.

The inspection and evaluation of the Chessman Dam and Chessman Saddle Dam under the National Dam Safety Program in 1980 revealed certain deficiencies in the project which resulted in recommendations for further investigations. The purpose of this investigation was to address those deficiencies and to determine the necessary requirements to provide a safe facility while optimizing the operation of this water supply.

The primary emphasis of the investigation addressed the stability of the Main Dam embankment and the adequacy of the project spillway. The geotechnical and stability analyses of the existing structure resulted in recommendations for modifications to the embankment. The hydrologic and hydraulic analyses established an appropriate spillway design flood.

This final report provides preliminary engineering design data for modifications, repairs and additions necessary to rehabilitate the Chessman Reservoir Project to acceptable design standards.

A detailed scope of services for this project is presented in Chapter of this report.

2.2 DESCRIPTION OF PROJECT

2.2.1 General

Chessman Reservoir is a municipal water supply storage project owned and operated by the City of Helena, Montana. It is located in Helena National Forest in Section 2, Township 8 North, Range 5 West, in Lewis and Clark County about 12 miles southwest of the City of Helena. The reservoir is perched atop a divide at the head of Beaver Creek in the upper Tenmile Creek drainage about 3.2 miles above the community of Rimini (Figure 1). Built in 1908, the reservoir is actually formed by two earthfill dams, a main embankment situated at the west end of the reservoir across Beaver Creek, and a smaller saddle dam situated 2,600 feet to the north on the divide between Beaver Creek and Buffalo Creek.

Chessman Main Dam, is 40 feet high, 440 feet long and impounds 1,630 acre-feet in 114 surface acres at a normal pool elevation of 99 feet (project datum). The height of the dam, impoundment volume of the reservoir, and the residential development in Rimini and along Tenmile Creek cause Chessman Main Dam to be classified as being of intermediate size with a high downstream hazard potential.

Chessman Saddle Dam is separated from the main embankment and contains the spillway structure for the project. The saddle dam is situated in a low divide separating the Beaver Creek drainage, in which the reservoir is located, from the Buffalo Creek drainage. Prior to construction of the project, no natural flow existed across the drainage divide. However, with construction of the dams and the creation of the reservoir, spillage of excess water was diverted to Buffalo Creek via the Saddle Dam spillway. The dam is 17 feet high and 105 feet long, with a spillway formed into the core wall. The total active storage impounded by the Saddle Dam is approximately 1,400 acre-feet.

2.2.2 Description of Dam and Appurtenances

Chessman Main Dam

Chessman Main Dam is an earth embankment with a concrete core wall extending the full length but not the full height of the dam. Field measurements verify the as-built dimensions as being 440 feet long, 40 feet high from the streambed to the crest, and 15 to 17 feet wide at the crest. The only outlet through the dam consists of a valved pipe used to regulate releases to the City of Helena's water distribution system. A dry well valve chamber provides access to the control valve at the center of the dam.

Chessman Saddle Dam

Chessman Saddle Dam is an earth embankment 17 feet high and 105 feet long with a concrete core wall extending the full length. The core wall is exposed along the upstream edge of the crest of the embankment and the full 10-foot width of the crest is paved with concrete. The spillway is formed into the core wall. A riprapped and paved chute 30 feet long directs the flow to heavily overgrown Buffalo Creek channel.

At one time a wooden bridge crossed the spillway to provide access along the east shore of the reservoir. After the bridge had deteriorated beyond repair, it was removed and substituted with a roadfill constructed about 200 feet upstream across the approach channel to the spillway. The causeway road, with an elevation approximately equal to the top of the Saddle Dam prohibits the spillway from functioning as designed and has an important bearing on the safety of the project.

Red Mountain Ditch

A water supply aqueduct known as the Red Mountain Ditch supplies water from the Banner Creek drainage to Chessman Reservoir. The ditch extends about 7½ miles from the southwest end of the reservoir near the dam, around Red Mountain, intercepting the runoff from Sally Bell Creek. Capacity of

the ditch is about 15 cfs and any excess flow will spill into the Beaver Creek drainage below Chessman Dam.

Design and Construction History

Chessman Main Dam and Chessman Saddle Dam were designed by Eugene Carroll for the Helena Water Works Company in January 1907. Construction is though to have been completed in 1908 although no documentation of that fact has been found. The only documentation found of the design and construction of the dams consists of one sheet labeled "Plan of Dam at the Chessman Reservoir", dated January, 1907 which is also noted "as-used", and some historical photographs of the construction. No information is available as to site investigations, material testing, construction inspection, borrow areas, or toe drains as related to original construction.

Inspection reports from the files of Helena National Forest mention repairs or modifications to the dam which may have been made in the period 1933 to 1935. Inspection of the dam revealed additional fill at the outlet, and drains at the outlet and downstream of the outlet which are modifications not shown on the as-used drawings. No documentation has been found concerning any repair work or modifications to the dam after 1935 nor is there any information concerning damage to the structure resulting from the earthquake.

3.0 REHABILITATION INVESTIGATION

3.1 INTRODUCTION

The adequacy of the project spillway located in the Saddle Dam, the identification, operation and notification of emergency situations and the stability of the existing Main Dam under static and seismic loads have been determined in the following technical analyses. The results of these analyses enable the City of Helena to determine the course of action necessary to rehabilitate the project and restore it to full operational capacity.

3.2 PROJECT INFLOW DESIGN FLOOD

3.2.1 Classification and Applicable Guidelines

Dam safety guidelines were enacted by the 92nd Congress following the Buffalo Creek disaster which occurred on February 26, 1972 and resulted in the loss of 118 lives and over 500 homes. Public Law 92-367 was legislated on August 8, 1972 and established a comprehensive national program for the inspection and regulation of dams. This law authorized the Secretary of the Army, through the Corps of Engineers, to assemble recommended guidelines and perform Phase I investigations of high hazard dams. Certain dams, such as those under the jurisdiction of the Bureau of Reclamation and Federal Power Commission, were excluded from the act.

The Chessman Project was inspected under the Phase I program and recommendations were made to perform a stability analysis of the main dam embankment and establish a spillway design flood and appropriate spillway modifications. The project is owned by the City of Helena and is operated as a municipal water supply under a special use permit from the Forest Service. Because the project must meet Forest Service standards only those guidelines and nomenclature will be used in this report. Note, that outside of small wording differences, the base guidelines used by the Forest Service are consistent with those used by other agencies, such as the Corps of Engineers and the Bureau of Reclamation. Detailed guidelines developed

by other agencies which cover areas not specifically addressed in the Forest Service guidelines will be used as appropriate.

Project classification^{1/} for dams on lands administered by the Forest Service are summarized below:

Administrative Classification of Dams

Class A Projects: Dams that are 100 feet or more in height, or impound 50,000 acre-feet or more of water.

Class B Projects: Dams that are 40 feet or more but less than 100 feet high, or impound 1,000 or more but less than 50,000 acre-feet of water.

Class C Projects: Dams that are 25 or more but less than 40 feet high, or impound 50 or more but less than 1,000 acre-feet of water.

Class D Projects: Dams that are less than 25 feet high and impound less than 50 acre-feet of water.

Hazard Potential Classification of Dams

Low Hazard. This applies to dams built in undeveloped areas where failure would result in minor economic loss and damage would be limited to undeveloped or agricultural lands, and improvements are not planned in the foreseeable future. Loss of life would be unlikely.

Moderate Hazard. This applies to dams built in areas where failure would result in appreciable economic loss with damage limited to improvements such as commercial and industrial structures, public utilities and transportation systems, and serious environmental damage. No urban development and no more than a small number of habitable structures is involved. Loss of life would be unlikely.

^{1/} Forest Service Region 1, General Procedures and Design Criteria for Water Storage and Transmission Projects, Missoula, Montana.

High Hazard. This applies to dams built in areas where failure would likely result in loss of life, or economic loss would be excessive. Generally, this would involve urban or community-type development with more than a small number of habitable structures.

Recommended Spillway Design Flood

<u>Hazard Potential</u>	<u>Administrative Class</u>	<u>Spillway Design Flood</u>
Low	A	$\frac{1}{2}$ PMF to PMF
	B	100 yr. to $\frac{1}{2}$ PMF
	C	50 yr. to 100 yr.
Moderate	A	PMF
	B	$\frac{1}{2}$ PMF to PMF
	C	100 yr. to $\frac{1}{2}$ PMF
High	A	PMF
	B	PMF
	C	$\frac{1}{2}$ PMF to PMF
	D	100 yr. to $\frac{1}{2}$ PMF

The Chessman Main Dam has a hydraulic height of 40 feet and impounds 1,990 acre-feet at the Saddle Dam crest elevation of 102 feet. Therefore, the project would carry a size classification of B.

Hazard classification of the Main Dam embankment which discharges to Beaver Creek would be rated high due to the town of Rimini which lies three miles downstream at the confluence of Beaver and Tenmile Creeks. Classification of the Saddle Dam embankment could be rated as moderate due to the distance and density of development downstream in the Buffalo Creek drainage. However, the potential for increased urban development exists which would result in a high hazard classification at a future date.

While there would appear to be room to argue for reduced hazard and possibly a size classification of the Saddle Dam embankment, it is not reasonable for the following explanation. The Chessman Main Dam has a crest elevation of 105 feet and depends on a concrete core wall for stability which has a crest elevation of 101 feet. The Main Dam must pass the Probable Maximum Flood (PMF) and not allow the reservoir to rise above elevation 101 feet. The Saddle Dam represents the existing and only possi-

ble spillway location and, therefore, it must be assigned the PMF as its Spillway Design Flood (SDF).

Figure 2 represents the analytical procedure for sizing spillways if an exception to the recommended spillway design flood magnitude is justified.

3.2.2 Probable Maximum Precipitation

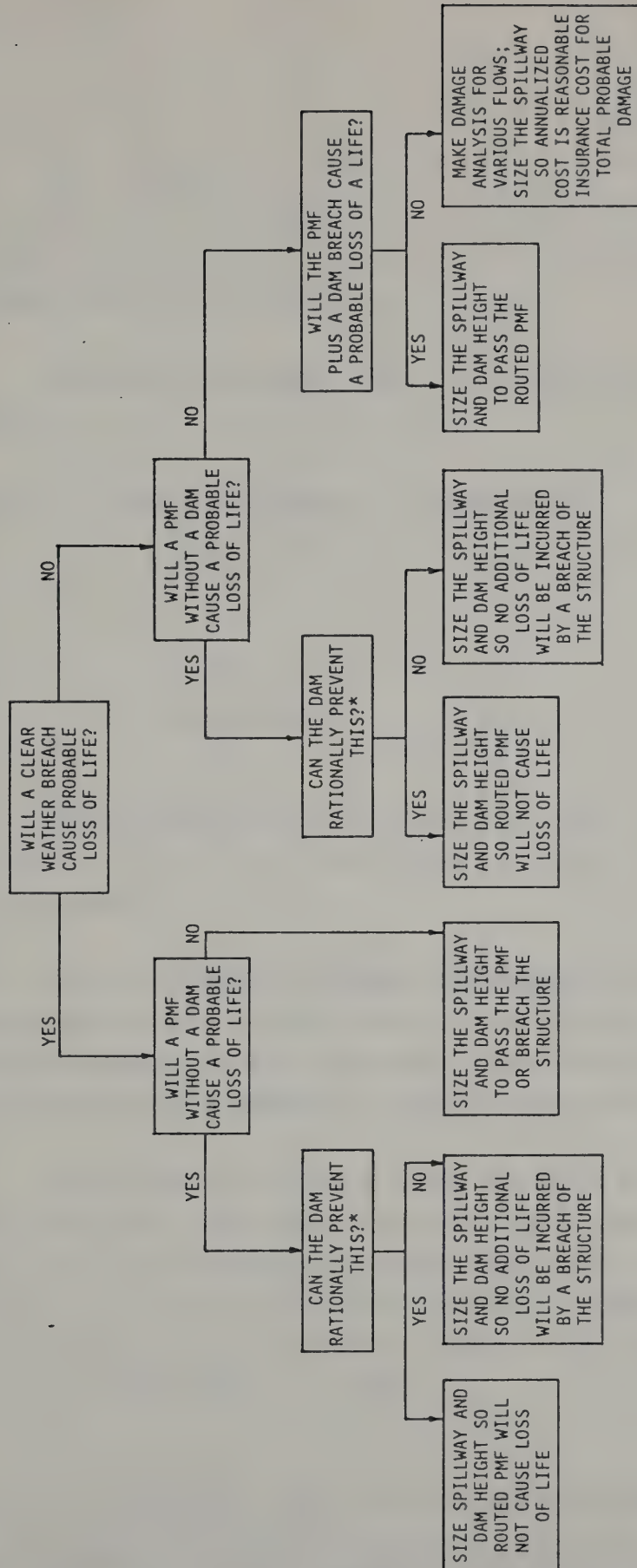
Detailed statistical data regarding the intensity and duration of severe storm events in the area including the Chessman Reservoir was not available for the Phase I investigation. Two sources of information were used in this study to develop the Probable Maximum Precipitation (PMP): 1) Storm transposition performed by Mr. Louis Schreiner, a meteorologist with the Bureau of Reclamation in Denver, and 2) Hydrometeorological Report (HMR) 55 developed by the National Weather Service. These investigations supercede Technical Paper 38 and the Interim Report used for Phase I investigations.

Determination of the correct PMP and resultant PMF must consider two basic hydrologic conditions. There are storm events of short duration and high intensity causing high flood crests and comparatively small flood volumes versus storm events of long duration and lesser intensity causing comparatively lower flood peaks and larger flood volumes. These two severe storm events at the Chessman Project could be described as a summer thunderstorm event occurring in July or August on a snow-free basis (short duration event) and a 2-3 day general storm event occurring in May or June and falling on a snow covered or saturated basin.

Snow survey records^{2/} at the Chessman Reservoir snow course indicates that during a 38-year period the water equivalent on May 1st has exceeded 7 inches on 3 occasions. Water equivalent on June 1st appears to be minimal with no readings recorded.

^{2/} Soil Conservation Service, Snow Survey Measurements for Montana 1922-1974, Bozeman, MT, 1975

ANALYTICAL PROCEDURE FOR SIZING DAM SPILLWAYS (Applicable to new or existing structures)



* "Rationally" includes physical possibility, practicality, economic considerations, etc.

Except where loss of life is a possibility, spillway size and dam height should be such that the cost is reasonable for the protection which is provided, including the dam.

FIGURE 2

Soils temperature data^{3/} is available for the Stemple Pass area which lies 40 miles to the north at a comparable elevation. Records indicate a frozen soil profile on May 1st on the average and only one year with a frozen profile at the end of May.

Three PMP case studies were investigated for this report and are shown below:

<u>Storm</u>	<u>Time of Occurrence</u>	<u>Frozen Ground</u>	<u>Snowmelt</u>	<u>Source</u>
A	May	Yes	Yes	Storm Transportation by Mr. Schreiner
B	June	No	No	HMR 55
C	July-August	Yes	No	HMR 55

Snowmelt was calculated using wind speed and temperature data published in HMR 43^{4/} and the Corps of Engineers Engineering Manual: Runoff from Snowmelt^{5/}.

Storm distribution patterns recommended by the Bureau of Reclamation were applied to each PMP event to provide for a storm pattern. Greatest hour increments of each storm were arranged in the reverse order of the unit hydrograph to provide for the greatest possible PMF peak.

Characteristics of PMP Storm A are shown on the Table 1 and Figure 3. This storm was considered to be an early season storm occurring during early May and falling on frozen ground and a snowpack. Total snowmelt during the 72-hour PMP period would equal approximately 5.5 inches and precipitation would equal 16.4 inches. The combination of rainfall and snowmelt would equal 21.9 inches.

^{3/} Soil Conservation Service, Soil Moisture Measurements, Montana 1949-1978, Bozeman, MT, 1978

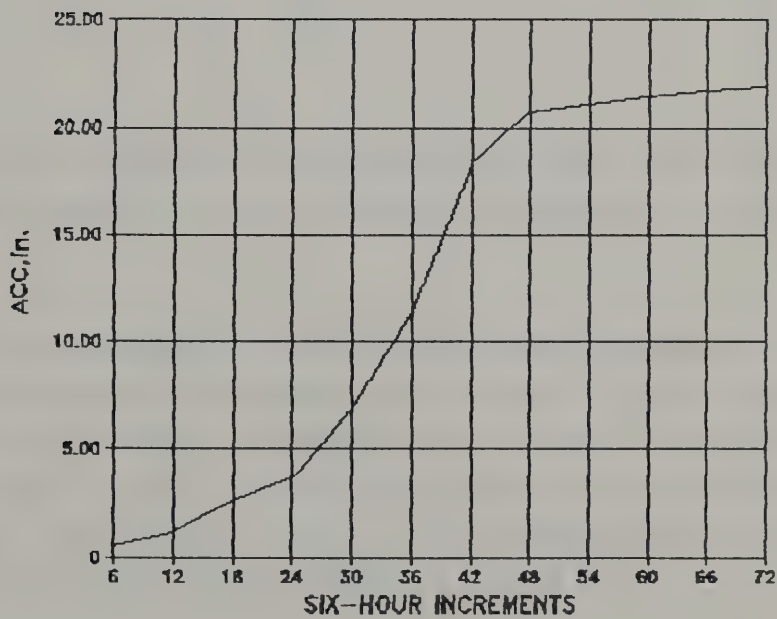
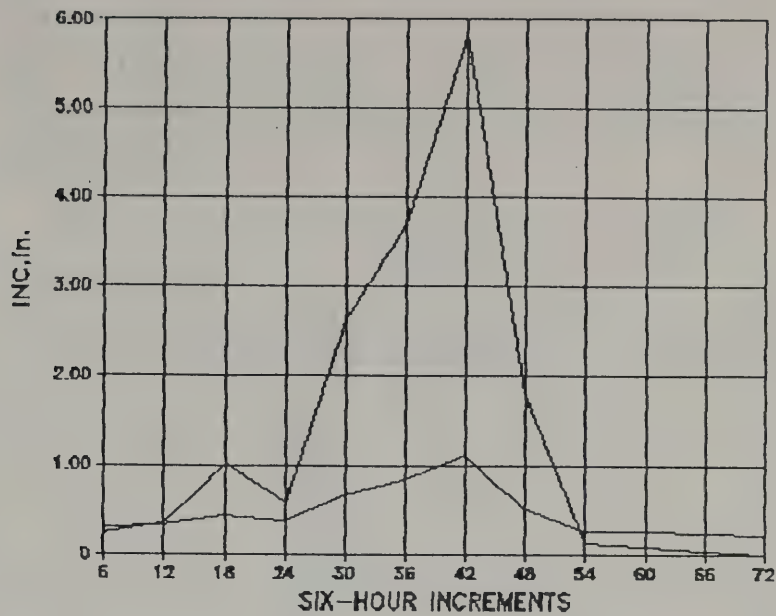
^{4/} Department of Commerce, Probable Maximum Precipitation Northwest States, HMR No. 43, Weather Bureau, Washington, D.C., November 1966

^{5/} U.S. - Army Corps of Engineers, Runoff From Snowmelt, EM 1110-2-1406, Washington, D.C., 1960

TABLE 1
PMP AND SNOWMELT TOTALS
PMP STORM A
SIX-HOUR INCREMENTS
CHESSMAN RESERVOIR PROJECT

HOUR	RANK	V, mph	P, inches	T, °F	SNOWMELT M6, inches	TOT STORM INC, in.	TOT STORM ACC, in
6.00	8.00	31.00	.24	42.00	.30	.54	.54
12.00	7.00	33.00	.35	42.25	.33	.68	1.21
18.00	5.00	37.00	1.04	43.00	.43	1.47	2.68
24.00	6.00	35.00	.60	42.50	.36	.96	3.64
30.00	3.00	41.00	2.65	44.50	.65	3.30	6.95
36.00	2.00	44.00	3.72	45.50	.83	4.55	11.50
42.00	1.00	47.00	5.83	46.00	1.09	6.92	18.42
48.00	4.00	39.00	1.76	43.50	.52	2.28	20.69
54.00	9.00	30.00	.12	41.50	.27	.39	21.09
60.00	10.00	29.00	.09	41.00	.25	.34	21.43
66.00	11.00	28.00	.04	40.50	.23	.27	21.70
72.00	12.00	27.00	.00	40.00	.21	.21	21.91
			16.44		5.47	21.91	

V - Incremental Wind Velocity
P - Incremental Precipitation
T, °F - Average Temperature During 6-hour Period
M6 - Incremental 6-hour Snowmelt
TOT STORM INC - Total Storm, Incremental
TOT STORM ACC - Total Storm, Accumulated



**CHESSMAN DAM
REHABILITATION INVESTIGATION**

PMP STORM A

FIGURE 3



MORRISON-MAIERLE, INC.
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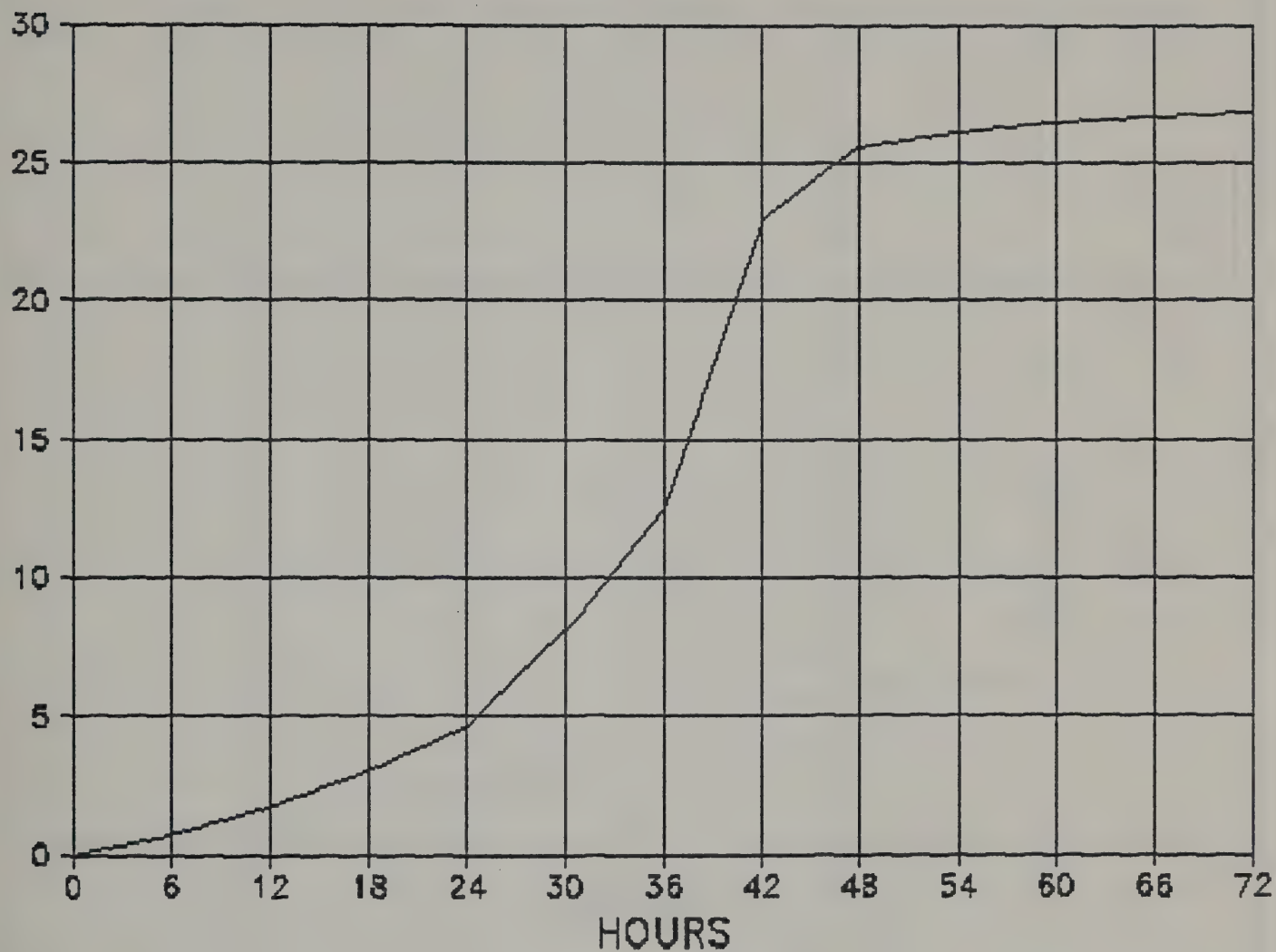
Storms B and C were developed from a draft copy of HMR 55 by Bureau of Reclamation personnel and transmitted to Morrison-Maierle in a letter dated January 19, 1984. Storm arrangements and expected months of occurrence were specified. The six-hour precipitation values for Storm B is shown below and illustrated on Figure 4.

STORM B		
<u>Hour</u>	<u>Total Storm Incremental (inches)</u>	<u>Total Storm Accumulated (inches)</u>
0	0.0	0.0
6	0.7	0.7
12	1.0	1.7
18	1.3	3.0
24	1.6	4.6
30	3.5	8.1
36	4.5	12.6
42	10.3	22.9
48	2.7	25.6
54	0.5	26.1
60	0.4	26.5
66	0.2	26.7
72	0.2	26.9

Storm B was considered to be a June event which would fall on a basin saturated by snowmelt but not covered by a snowpack of significant water content.

Storm C is considered to be a severe local thunderstorm event occurring in July or August and lasting for six hours. A storm distribution was recommended by the Bureau of Reclamation and used in this analyses. This storm would occur during a period of the year when basin infiltration rates could be at or above minimum values. The incremental and accumulated storm values are shown below and illustrated on Figure 5.

ACC PRECIP, inches



CHESSMAN DAM
REHABILITATION INVESTIGATION

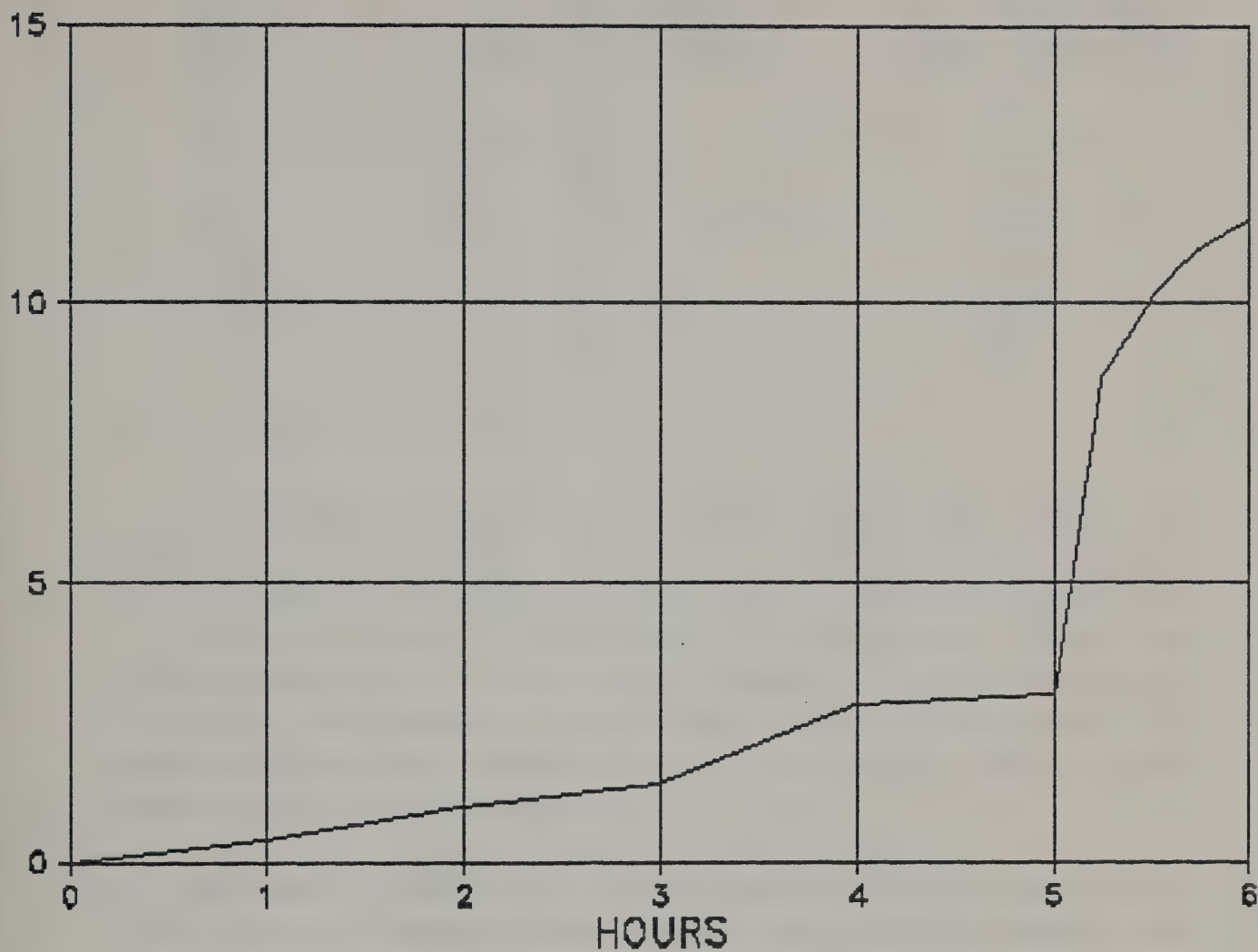
PMP STORM B

FIGURE 4



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ACC PRECIP, inches



**CHESSMAN DAM
REHABILITATION INVESTIGATION**

PMP STORM C

FIGURE 5



MORRISON-MAIERLE, INC.
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STORM C

<u>Hour</u>	<u>Total Storm Incremental (inches)</u>	<u>Total Storm Accumulated (inches)</u>
0	0.0	0.0
1	0.4	0.4
2	0.6	1.0
3	0.4	1.4
4	1.4	2.8
5	0.2	3.0
5.25	5.7	8.7
5.50	1.4	1.4
5.75	0.9	11.0
6.00	0.5	11.5

3.2.3 Basin Characteristics

The drainage basin contributing to the Chessman Dams includes 1.15 square miles of mountainous forest and 0.18 square miles of reservoir surface. Rainfall and snowmelt on the reservoir surface contributes instantaneously to reservoir inflow without significant losses. Rainfall and snowmelt occurring on the land units is subject to losses such as infiltration and interception and must travel overland to the reservoir. The characteristics of the overland travel of runoff within a basin is represented by the Time of Concentration.

The Time of Concentration (T_c) is defined as the time required for runoff to travel from the furthest point in the basin to the project site. It is influenced by factors such as length and slope of stream channel, type of vegetative cover, and soil types. Time of Concentration can be determined from precipitation and stream gaging data when sites are located within the project basin (or similar basins) or by empirical formulas. Because there are no hourly precipitation and stream gaging stations located in the project basin or vicinity, the T_c must be determined by empirical formulas.

For a known drainage area and PMP event of given duration, the T_c affects the design flood as follows:

- ° a shorter T_c will produce a greater peak flow
- ° a longer T_c will produce a lesser peak flow
- ° the runoff volume will be nearly identical

The following information was used to determine the T_c and unit hydrographs for the Chessman basin.

Location: Township 8 North, Range 5 West, Sections 1,2,11,12,14
Latitude 112° - 11' Longitude 46° - 28'
Lewis and Clark County, Montana

Drainage Area: Land Mass: 736 acres = 1.15 square miles
Reservoir Surface: 114 acres = 0.18 square miles
Total 850 acres: 1.33 square miles

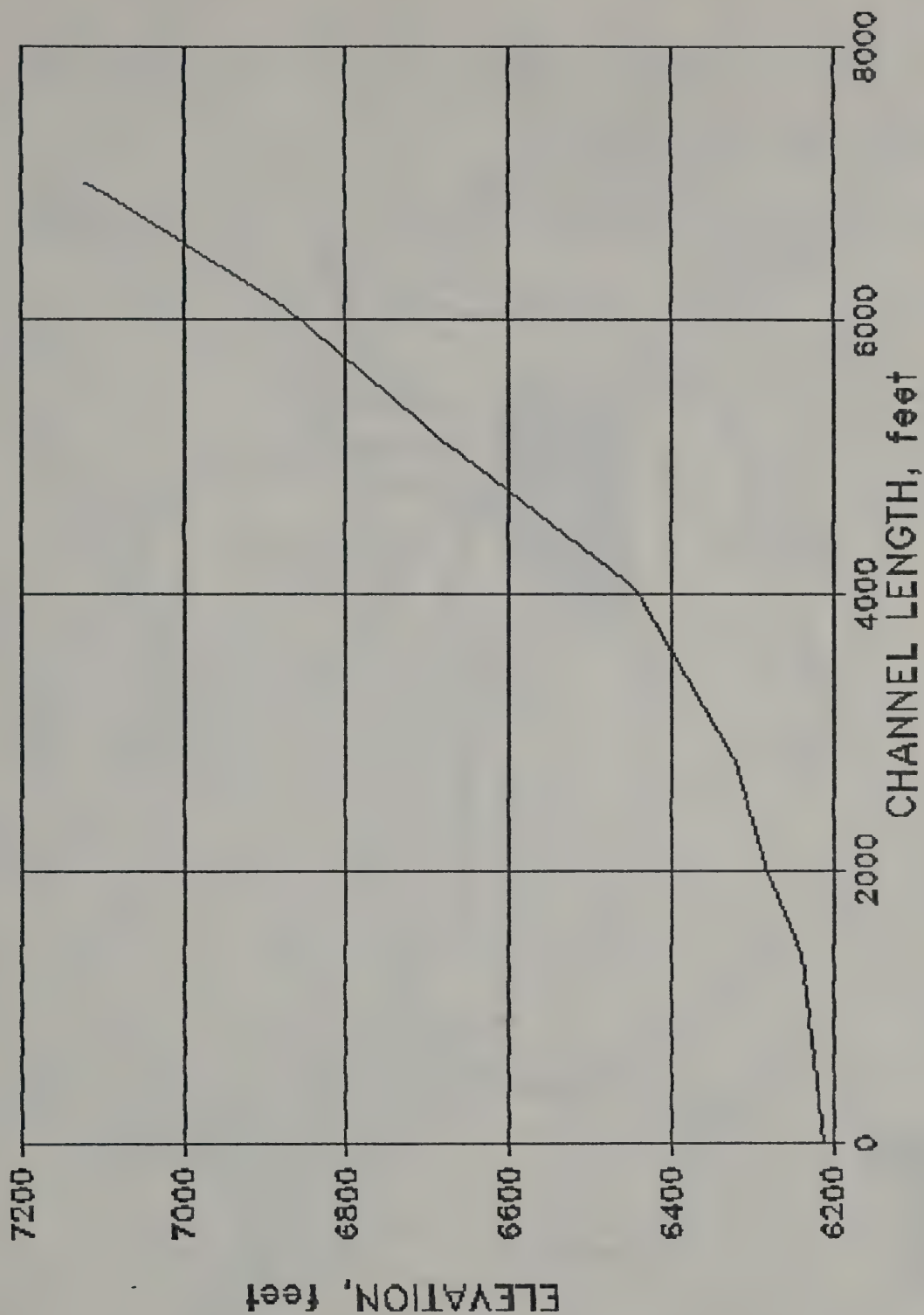
Land Cover:	Marsh	34 acres
	Open Range	39
	Forest	<u>663</u>
	Total:	736 acres

Elevation Range:	At Dam	6200 feet
	At Ridge	7120 feet

A profile of the main drainage channel is shown on Figure 6 and on outline of the drainage basin is shown on Figure 7.

Special characteristics of the Chessman Reservoir basin:

- ° 78 percent forest cover
- ° 13 percent reservoir surface
- ° Average channel slope 13 percent

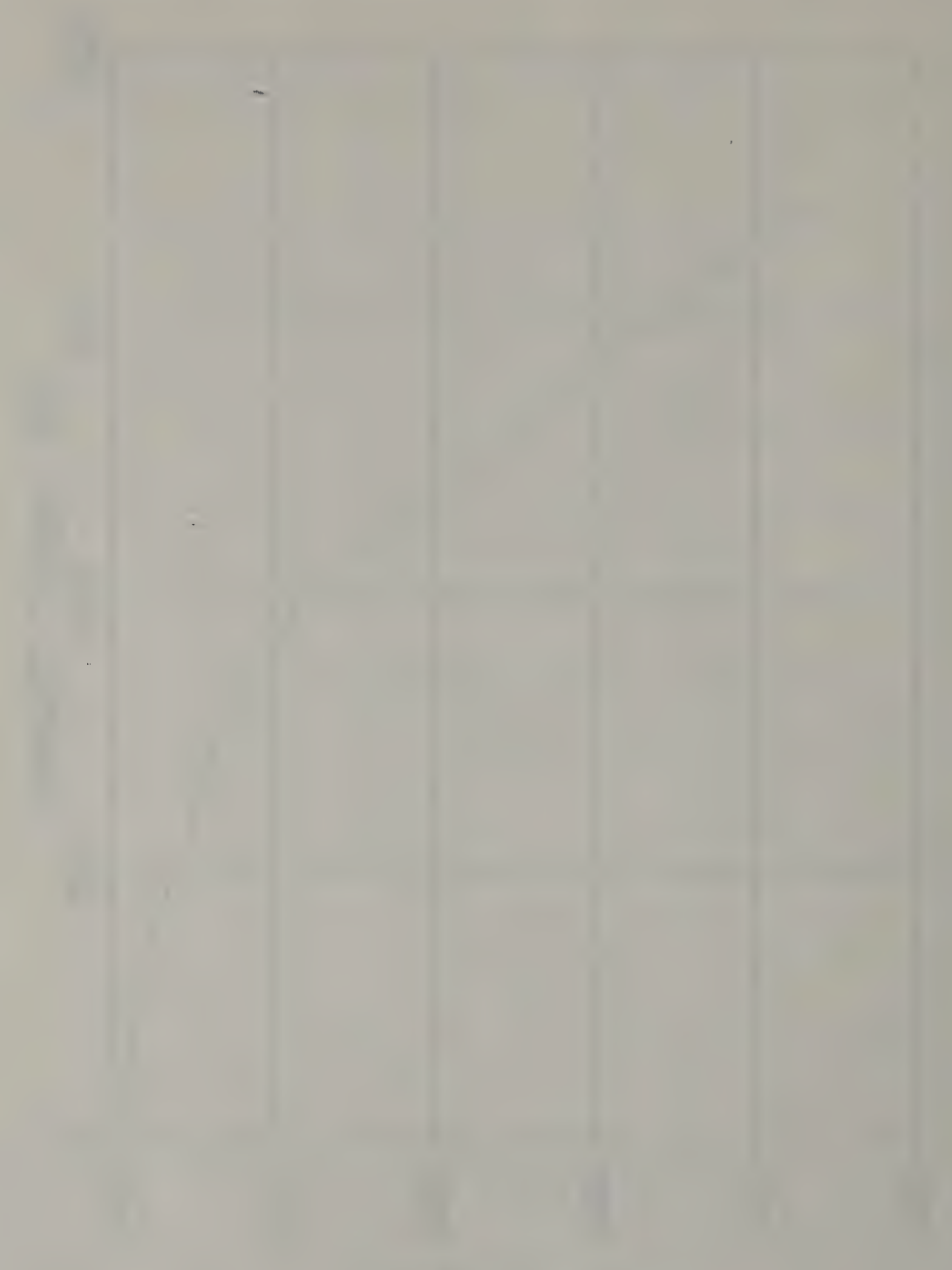


**CHESMAN DAM
REHABILITATION INVESTIGATION
MAIN CHANNEL PROFILE
CHESMAN DAM DRAINAGE**

FIGURE 6



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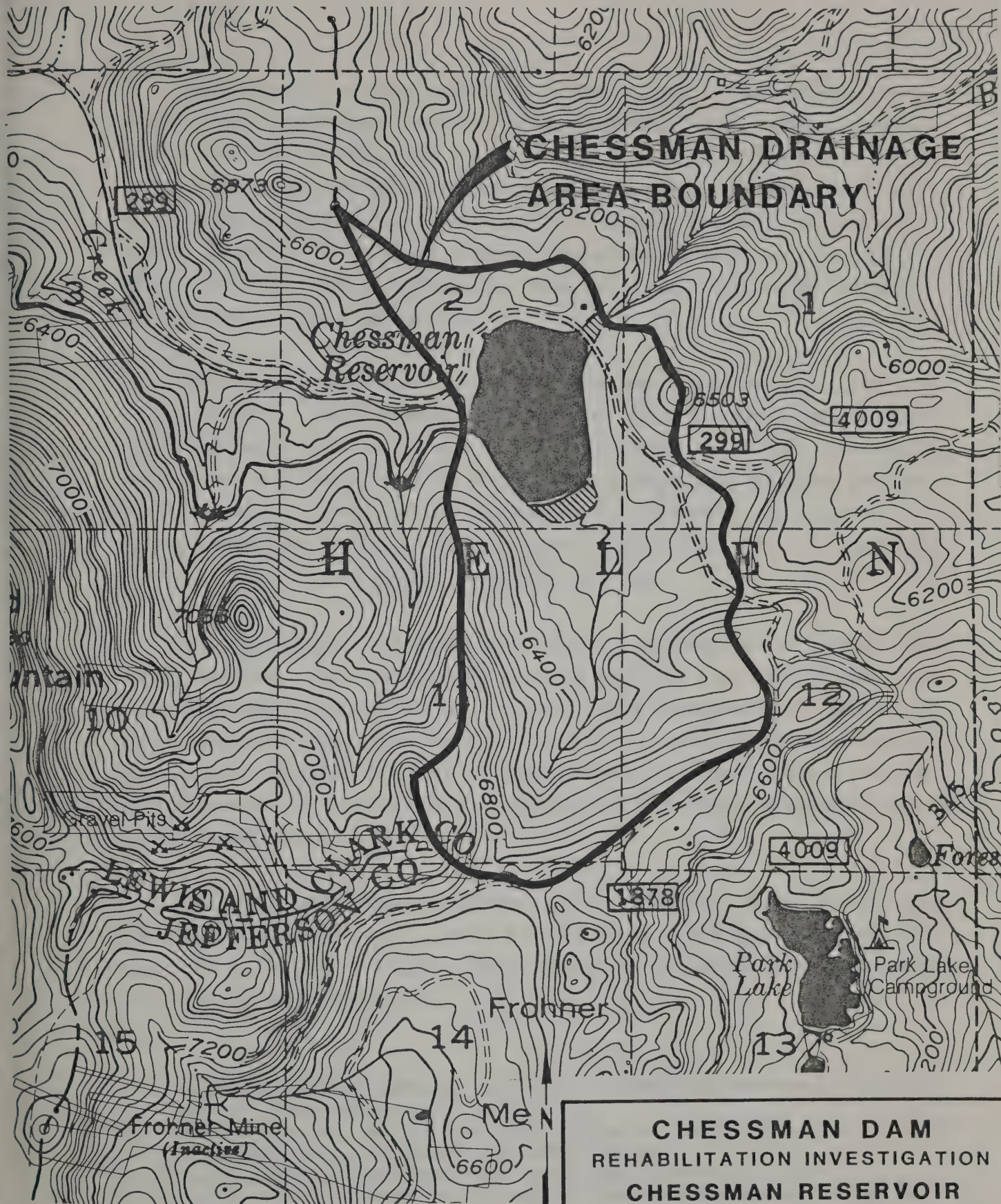


Distance vs. Time

Graph showing the relationship between Distance and Time.

The curve starts at the origin (0,0) and increases, showing a decreasing rate of change (concave down).

The x-axis is labeled 'Time' and the y-axis is labeled 'Distance'.



1000 0 1000 2000 3000 4000 5000 6000 7000

SCALE (feet)

**CHESSMAN DAM
REHABILITATION INVESTIGATION
CHESSMAN RESERVOIR
DRAINAGE BASIN MAP**

FIGURE 7



MORRISON-MAIERLE, INC.
CONSULTING ENGINEERS

Under dry and unfrozen conditions a forested basin can have good water holding and retardance characteristics, however, most extreme rainfall events will occur in May when a considerable snowpack and frozen ground can exist. Storms occurring in June would probably precipitate on a basin with soil profiles completely saturated from earlier rainfall and snowmelt. Storms occurring in July and August were assumed to occur on soil profiles with a minimum intake value of 0.15 inch per hour. This value is typical of hydrologic class B soils.

Time of Concentration for the land mass was estimated using the following methods: Soil Conservation Service (SCS)^{6/}, Ramser-Kerby equation^{7/}, Kirpich equation^{8/}, and Corps of Engineers equation^{9/}. Estimates ranged from 0.26 hours to 0.81 hours. The Ramser-Kerby equation was used in this analysis and predicted a value $T_c = 0.55$ hours. It uses the following parameters: channel length, channel slope, Manning's 'n', length of overland flow path and slope of overland flow path.

Unit hydrographs are based upon the T_c , drainage area, and unit of runoff and time. Unit hydrographs are used with the time distributed storm event to produce the inflow design flood. Unit hydrographs are defined as the hydrographs that would result from 1 unit (inch) of runoff occurring over a unit duration (varies). Dimensionless unit hydrograph procedures were developed by the SCS and are conventionally accepted methodology used by all hydrologists.

Unit hydrographs were developed using procedures found in the SCS Hydrology Handbook^{10/}, a T_c equal to 0.55 hours and a time increment (D) equal to 0.1 hours. A short time increment is required to accurately subdivide the unit hydrograph.

^{6/} Soil Conservation Service, National Engineering Handbook, Hydrology Section 4, Washington, D.C., 1971.

^{7/} American Society of Agricultural Engineers, Hydrologic Modeling of Small Watersheds, St. Joseph, MI, 1982, 533 pp.

^{8/} Bureau of Reclamation, Design of Small Dams, Denver, CO, 1977.

^{9/} Linsley, Kohler, Paulus, Hydrology for Engineers, Mc-Graw - Hill, New York, 1958.

^{10/} See note 6.

The corresponding unitgraph has a time to peak, time of base and peak equal to respectively; 0.40 hours, 2.0 hours and 1400 cfs.

It should be noted that the unitgraph for a particular basin is not constant, changing basin conditions will continually change the time of concentration and resulting unitgraph. A sensitivity analysis was performed which varied the T_c from 75 percent to 125 percent of the selected value. The resulting unitgraph peaks varied 128 percent and 82 percent respectively. Because the project has significant storage capacity, findings in the inflow design flood routing indicated that the spillway requirements were not sensitive to the time of concentration (over a reasonable range).

Individual unit hydrograph data is shown on Table 2 and each unitgraph on Figure 8.

TABLE 2
SOIL CONSERVATION SERVICE UNIT HYDROGRAPH PROCEDURES

DRAINAGE BASIN NAME = CHESSMAN DAM

DRAINAGE AREA (A), square miles 1.15

75% Tc

124% Tc

TIME OF CONCENTRATION (Tc), hour .58

.44

.73

TRIANGULAR UNIT HYDROGRAPH

UNIT TIME INCREMENT (D), hours = .10

UNIT RUNOFF (Q), inches = 1.00

TIME TO PEAK (Tp), hours = .40

.31

.49

TIME OF BASE (Tb), hours = 1.06

.83

1.29

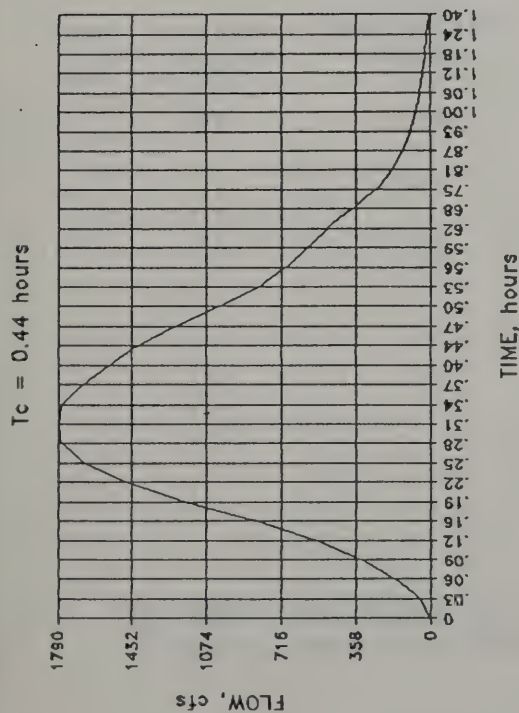
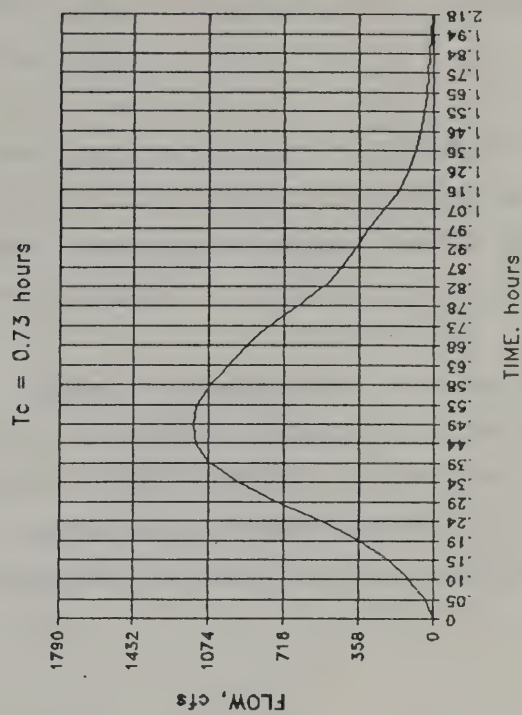
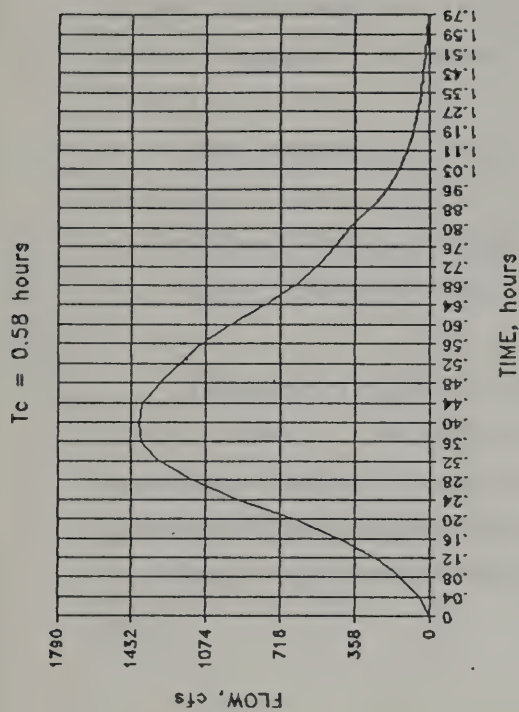
HYDROGRAPH PEAK FLOW (qp), cfs = 1398.49

1789.71

1147.63

CURVILINEAR FIT FOR UNIT HYDROGRAPH

TIME RATIOS (t/tp)	FLOW RATIOS (q/qp)	TIME hours	FLOW cfs	TIME hours	FLOW cfs	TIME hours	FLOW cfs
.00	.00	.00	.00	.00	.00	.00	.00
.10	.83	.04	41.95	.03	53.69	.05	34.43
.20	.10	.08	139.85	.06	178.97	.10	114.76
.30	.19	.12	265.71	.09	340.05	.15	218.05
.40	.31	.16	433.53	.12	554.81	.19	355.76
.50	.47	.20	657.29	.16	841.16	.24	539.39
.60	.66	.24	923.01	.19	1181.21	.29	757.44
.70	.82	.28	1146.76	.22	1467.56	.34	941.06
.80	.93	.32	1300.60	.25	1664.43	.39	1067.29
.90	.99	.36	1384.51	.28	1771.81	.44	1136.15
1.00	1.00	.40	1398.49	.31	1789.71	.49	1147.63
1.10	.99	.44	1384.51	.34	1771.81	.53	1136.15
1.20	.93	.48	1300.60	.37	1664.43	.58	1067.29
1.30	.86	.52	1202.70	.40	1539.15	.63	986.96
1.40	.78	.56	1090.82	.44	1395.97	.68	895.15
1.50	.68	.60	950.97	.47	1217.00	.73	780.39
1.60	.56	.64	783.16	.50	1002.24	.78	642.67
1.70	.46	.68	643.31	.53	823.27	.82	527.91
1.80	.39	.72	545.41	.56	697.99	.87	447.58
1.90	.33	.76	461.50	.59	590.60	.92	378.72
2.00	.28	.80	391.58	.62	501.12	.97	321.34
2.20	.21	.88	289.49	.68	370.47	1.87	237.56
2.40	.15	.96	205.58	.75	263.09	1.16	168.70
2.60	.11	1.83	149.64	.81	191.50	1.26	122.80
2.80	.08	1.11	107.68	.87	137.81	1.36	88.37
3.00	.06	1.19	76.92	.93	98.43	1.46	63.12
3.20	.04	1.27	55.94	1.00	71.59	1.55	45.91
3.40	.83	1.35	40.56	1.06	51.98	1.65	33.28
3.60	.02	1.43	29.37	1.12	37.58	1.75	24.10
3.80	.02	1.51	20.98	1.18	26.85	1.84	17.21
4.00	.01	1.59	15.38	1.24	19.69	1.94	12.62
4.50	.81	1.79	6.99	1.48	8.95	2.18	5.74
5.00	.00	1.99	.00		.00		.00



CHESMAN DAM REHABILITATION INVESTIGATION

SCS UNIT HYDROGRAPHS

FIGURE 8



MORRISON-MAIERLE, INC.
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3-11

3.2.4 Inflow Design Flood Hydrographs

Storm precipitation, unit hydrographs and reservoir storage - discharge data was used as input to the Corps of Engineers HEC-1 Flood Hydrograph Program^{11/} to develop the inflow design flood.

The 72-hour storms, A and B, have long periods prior to the peak which would fill the reservoir to the spillway crest and cause discharges prior to the peak inflow. Base flows would continue for a long period following the storm, therefore, the spillway may be used for a period of 3 to 4 days.

Conversely, the 6-hour local storm, C would occur in a relatively short period. The reservoir was considered to have been filled by a prior storm to the spillway crest. The peak inflow occurs rather quickly and large spillway discharges would be limited to a 24-hour period. Therefore, spillway discharges from a storm occurring 24 hours earlier would be minimal.

Each inflow flood has different characteristics which are summerized below:

Storm	Duration (hours)	Total Precipitation (inches)	Total Snowmelt (inches)	Total Loss (inches)	Total Runoff (inches)	Total Runoff (A-F)	Peak Inflow (cfs)
A	72	16.4	5.5	0	21.9	1550	1380
B	72	26.9	0.0	0	26.9	1905	5400
C	6	11.5	0.0	0.90	10.6	760	8640

^{11/} U.S. Army Corps of Engineers, HEC-1 Flood Hydrograph Program, HEC Center, Davis, CA, 1980

3.3 PROJECT ROUTING

3.3.1 Starting Reservoir Criteria

Chessman Reservoir is filled by runoff from a 1.3 square mile drainage area and the Red Mountain Ditch with a capacity of 15 cfs. Operational constraints require that diversions to the ditch occur only during periods of excess stream flows. Consequently, the Red Mountain Ditch is cleared of snow as early as possible in the runoff season and used to fill the reservoir.

Present project configuration consists of an outlet works of limited capacity (25-30 cfs) and a spillway with an uncontrolled crest. Under worst case assumptions a critical PMP storm would occur with the reservoir at the spillway crest elevation. The reservoir could be filled by prior storms and/or snowmelt and diversions to the Red Mountain Ditch. Because the project is operated for water storage, it is not unlikely that it would be full.

The following base flows were used to account for runoff occurring prior to the peak storm intervals:

	<u>Storm A</u>	<u>Storm B</u>	<u>Storm C</u>
Baseflow (cfs)	140	140	15

Starting reservoir elevations were established using these flows and the spillway rating curve. The Forest Service causeway was assumed to be removed.

3.3.2 Embankment Freeboard

Primary consideration for embankment freeboard would be allowance for wave runup and to insure adequate safety factors for foundation, embankment

and corewall conditions. Wave runup was predicted using methods presented by the Corps of Engineers^{12/} and wind speeds from Hydrometeorological Report HMR 43^{13/}.

Spring winds on an open reservoir present the worst case at the Chessman project with a fastest mile speed of 60 mph and a maximum one hour equal to 35 mph.

The effective fetch at the main dam is 0.37 mile and is in alignment with westerly winds. The critical design wind parameters equal a velocity of 51 mph and a time of 8 minutes. The wave period would equal 2.1 seconds.

The calculated wave runup on the existing flagstone riprap which has a relative smooth slope would equal 3.6 feet. Maximum runup on an embankment with dumped riprap protection which would have a rougher slope would equal 2.3 feet.

3.3.3 Existing Spillway Adequacy

Each storm was routed through the existing spillway under the conditions and criteria discussed prior and summarized below.

Existing Spillway				
Storm	Crest Elevation (ft.)	Peak Inflow (cfs)	Peak Outflow (cfs)	Maximum Res. Elevation (ft.)
A	97	1380	580	101.6
B	97	5400	1132	104.1
C	97	8640	860	102.9

^{12/} U.S. Army Corps of Engineers, Engineering Technical Letter 1110-2-221, Washington, D.C., November, 1976

^{13/} See Note 4

Key Elevations:

Spillway Crest - 97.0 feet

Crest of Saddle Dam - 102.0 feet

Crest of Main Dam - 105.0 feet

Top of Core Wall Main Dam - 101.0 feet

The spillway rating curve was developed using five field cross sections and the Corps of Engineers HEC-2 Water Surface Profiles Program^{14/}.

These sections began at the location of the proposed Forest Service road relocation 300 feet downstream of the spillway and extend into the reservoir. Because the reservoir is fairly shallow in front of the saddle dam there are approach losses. Peak outflow at the crest elevation of the saddle dam equals 660 cfs.

Note that each event would overtop the corewall elevation at the main dam which could result in the failure of the main dam embankment.

Storm B presents the worst case for the spillway design at the Chessman Project and will be used as the Spillway Design Flood (SDF) discussed in following sections. While it does not have the peak inflow of the Storm C event, its combination of peak inflow and volume present the worst routing condition.

3.3.4 Spillway Alternatives

Feasible emergency spillway sites at the Chessman Project are limited to the Saddle Dam. A concrete chute spillway could be installed on the Main Dam but it would require extensive rehabilitation of foundation embankment conditions and be extremely costly.

^{14/} U.S. Army Corps of Engineers, HEC-2 Water Surface Profiles Computer Program, HEC Center, Davis, California, 1981

Because of the magnitude of the spillway design flood and desired operational elevation at 99.0 feet, all of the available crest width at the Saddle Dam location will be required. Crest width between abutments of quartz-monzonite bedrock equals approximately 105 feet.

Rating curves were developed for 105 foot crest width at two elevations; 97 feet and 99 feet. These curves were based upon a spillway shape functioning as a broad-crested weir and were backwatered into the reservoir to account for approach losses. Rating curves for the two spillway alternatives are shown on Figure 9.

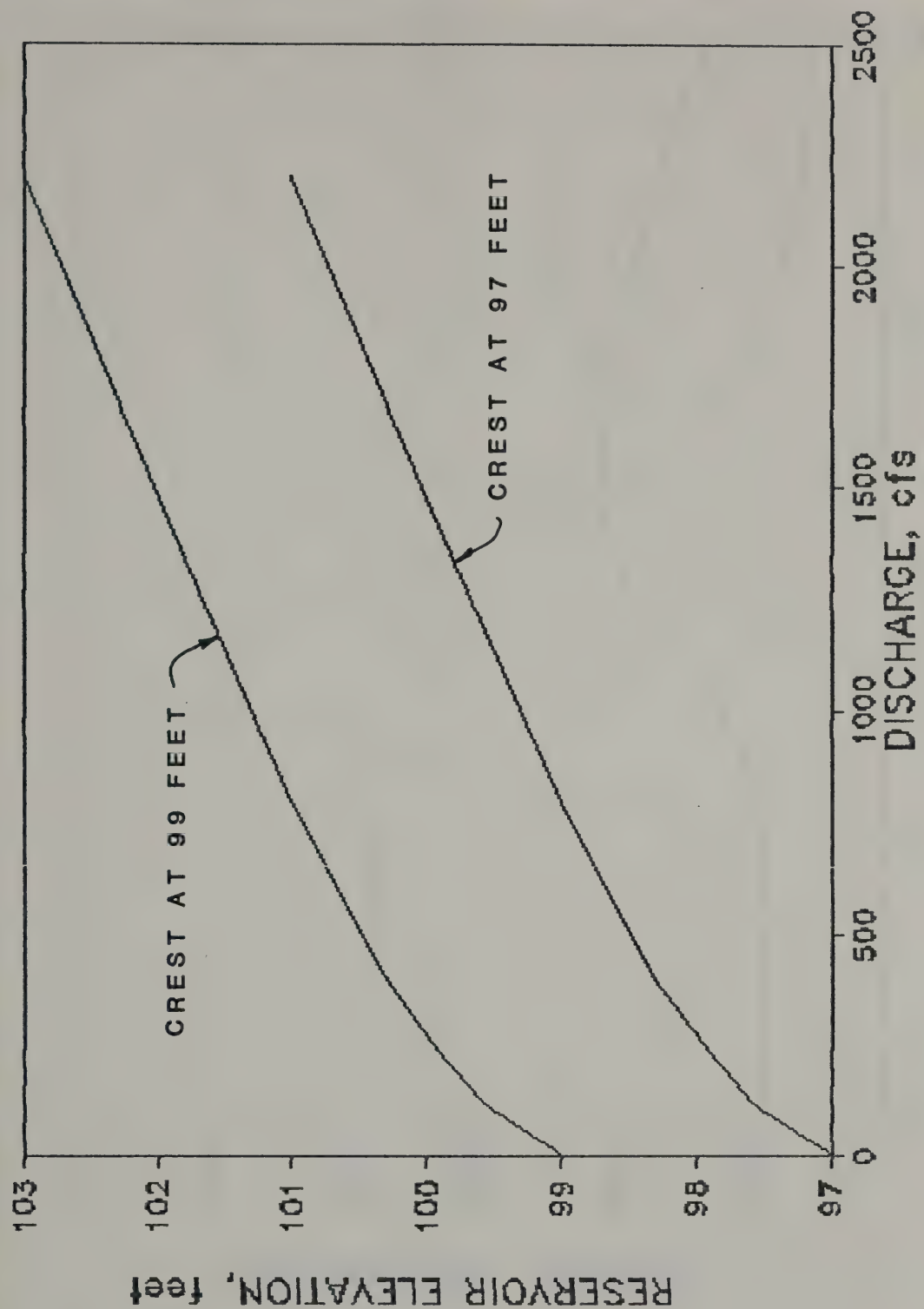
Peak outflow at elevation of 101 feet (cutoff wall) and with a spillway crest at 97 and 99 feet would equal 2140 and 800 cfs, respectively.

3.3.5 Routing of Spillway Alternatives

Routings of the two spillway alternatives were performed using conditions and criteria discussed in previous sections for the Spillway Design Flood (SDF). Peak inflow equals 5400 cfs during Storm B and a sufficient base flow occurs prior to the peak to fill the reservoir and use the uncontrolled spillway.

Routing the SDF requires 3.9 feet of head above the crest elevation when using a broad-crested weir section. Within a reasonable range of spillway crest elevations the spillway must be capable of discharging 2100 cfs.

Results of the routing analysis are summarized below and presented on Figure 10.

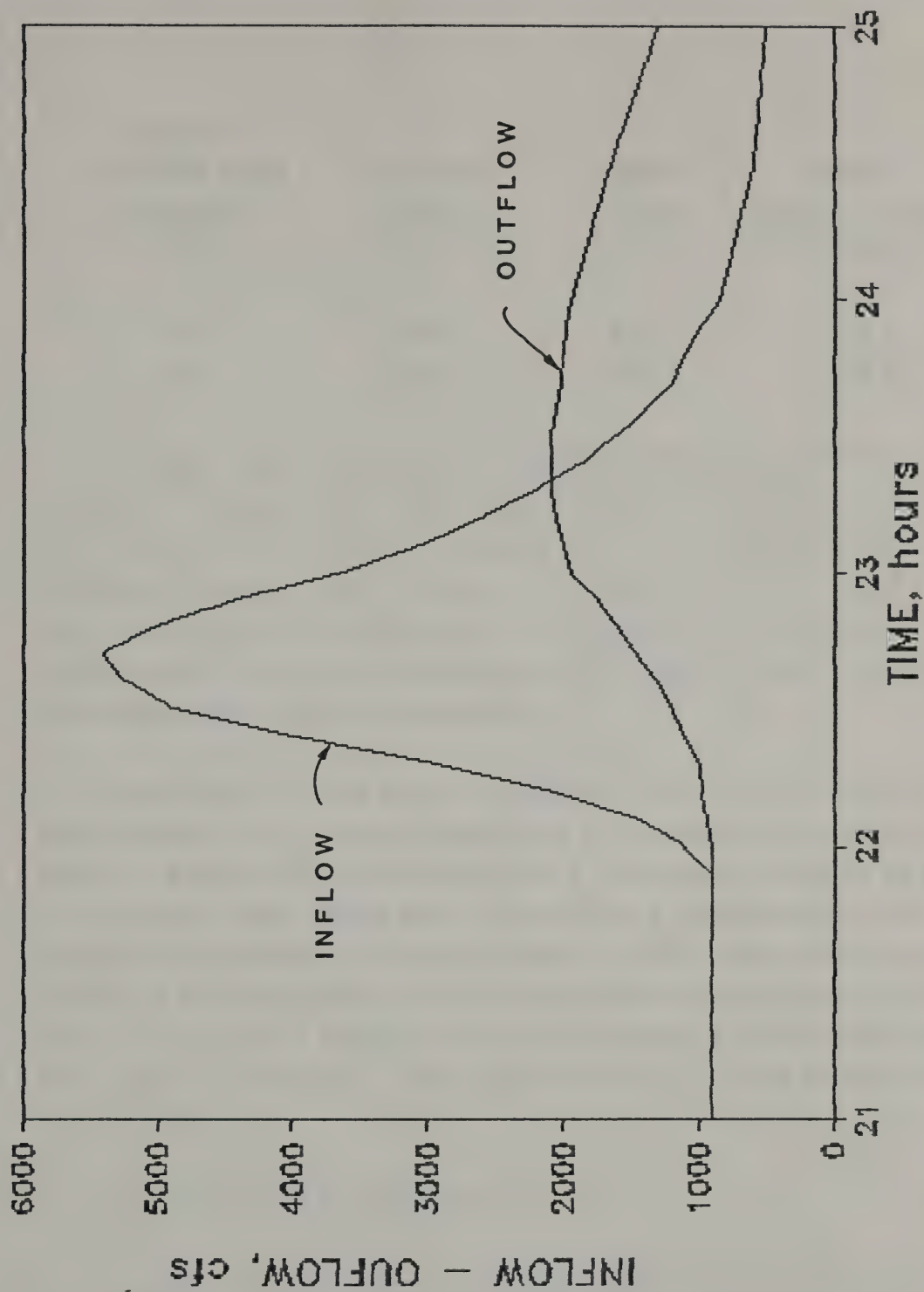


**CHESSMAN DAM
REHABILITATION INVESTIGATION
PROPOSED SPILLWAY RATINGS**

FIGURE 9



MORRISON-MAIERLE, INC.
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**CHESMAN DAM
REHABILITATION INVESTIGATION**

SPILLWAY DESIGN FLOOD

FIGURE 10



MORRISON-MAIERLE, INC.
CONSULTING ENGINEERS

Proposed Spillway Crest <u>Elevation</u> (ft.)	Maximum <u>Inflow</u> (cfs)	Maximum <u>Outflow</u> (cfs)	Maximum <u>Reservoir Elevation</u> (ft.)
97	5400	2100	100.9
99	5400	2100	102.9

An ogee shape spillway with greater discharge efficiency was investigated. The ogee shape which approximates the profile of the under nappe of a jet flow over a sharp-crested weir is the ideal form for obtaining optimum discharges. This shape would require 3.4 feet of head to route the SDF. Because an ogee shape would be expensive to construct with a relatively small increase in performance (0.5 feet of head reduction), it was eliminated from further consideration.

Sensitivity of the project routing to the time of concentration (T_c) was checked using T_c values from 75 to 125 percent of the selected T_c (0.55 hours). Peak inflow with 75 percent T_c increased from 5400 to 6200 cfs or 15 percent. Peak inflow with 125 percent T_c decreased to 4900 cfs or 10 percent. The effect of either change on the routed outflow or spillway discharge was negligible. The routed outflow at 75 percent T_c increased 40 cfs or less than 2 percent and at 125 percent T_c the outflow decreased a like amount. Therefore, the project is not sensitive to the basin time of concentration.

3.3.6 Selection of Spillway Alternative

Two spillway alternatives are proposed with crest elevations of 97 and 99 feet.

Crest Elevation at 97 feet:

- Routes SDF with existing corewall
- Reduces maximum operating elevation 2 feet and eliminates

201 acre-feet or 65,000,000 gallons of storage.

Crest Elevation at 99 feet:

- ° Requires extension of existing corewall to route SDF
- ° Maintains maximum operating elevation at 99 feet

Final recommendations on the spillway shape, size and crest elevation are made following structural considerations (Chapter 3.5.4) and geotechnical considerations (Chapter 3.5.5) and cost estimates (3.5.7).

3.4 GEOTECHNICAL EVALUATION

The purpose of the geotechnical investigation is to determine the stability of the Chessman Dam under all possible conditions of reservoir storage levels, seismic loads and seepage conditions. The investigations conducted by Northern Engineering and Testing, Inc., specifically covered the following scope:

1. Conduct sufficient field and laboratory investigations and engineering analysis to determine the stability of the Chessman embankments under static and seismic load conditions.
2. If the stability of the Chessman embankments does not meet current safety criteria, provide alternates for modification.
3. Provide sufficient information for construction cost estimates.
4. Provide an engineering report of findings with recommendations for rehabilitation.

A detailed geotechnical report^{15/} was prepared by Northern Engineering and Testing and is paraphrased in the following sections through 3.4.6 with portions also included in the APPENDIX. Additionally, supporting documents from Northern Engineering and Testing were prepared in response to questions raised during Chessman Dam Steering Committee meetings. ^{16/} ^{17/} These documents define liquefaction potential, strength parameters in stability analyses and compaction grouting effectiveness.

Project History: Construction of the Main and Saddle Dams was completed in 1908. Only limited information on design and construction is available, including one design drawing and several photographs. Construction of the embankments was accomplished with horse and wagons. The borrow sources were apparently located near each dam. The original design drawing for the Main Dam shows a 36 inch wide concrete cutoff wall extending below the embankment, but the depth of the bottom of the cutoff is not specified.

At the Main Dam there is a toe berm on the embankment and two cast-iron drains discharge from the downstream face. These features are not shown on the original design drawing, and it is unclear when they were constructed or the conditions that prompted their construction.

Seismicity: The Chessman site is located in the Intermountain Seismic Belt of Western Montana ^{18/} and has a history of past seismic activity. The

^{15/} Northern Engineering and Testing, "Report of Geotechnical Investigation Chessman Dam - Helena, Montana" to Morrison-Maierle, Inc., December 1, 1983.

^{16/} Northern Engineering and Testing, Inc., "Geotechnical Design Data - Chessman Dam Project" to Morrison-Maierle, Inc., June 15, 1984.

^{17/} Northern Engineering and Testing Inc., "Chessman Dam Rehabilitation Project" to Morrison-Maierle, Inc., September 10, 1984.

^{18/} Qamar, Anthony I., and Stickney, Michael C. "Montana Earthquakes, 1869 to 1979, Historical Seismicity and Earthquake Hazard,": Memoir 51, Montana Bureau of Mines and Geology and the University of Montana, Butte, Montana, 1983.

largest and most destructive events in this area occurred during October and November, 1935. Within this time period, two major shocks measuring about 6.3 and 6.0 on the Richter Magnitude Scale, occurred on October 18 and 31, 1935, respectively. Seismological evidence indicates the source of this activity was probably movement along the Prickly Pear fault zone ^{19/}. The epicenters of these earthquakes were located about 4 miles north of Helena. There is no record of the performance of either dam during this earthquake sequence, however, estimates of operating conditions from later years indicates the reservoir was relatively full and from reports and interviews, the toe berm did not exist in 1935. ^{20/} Apparently, no major damage occurred to either embankment, although damage in the City of Helena and surrounding area was extensive. A map and table of all recorded earthquakes in the Chessman area is included in the APPENDIX.

An earthquake with a magnitude in excess of 6 on the Richter Scale, centered about 190 miles southwest of Helena, Montana, in October 28, 1983. The National Earthquake Center has no reports of damage in the Helena area, nor did an inspection of the Chessman Dam indicate any activity as a result of this earthquake.

3.4.1 Field Investigations

Field investigations were conducted in three phases, utilizing backhoe test pits, and truck-mounted drills using hollowstem augers or rotary drilling techniques. During the first phase, backhoe test pits were excavated along the crest of the Main Dam, and on the abutment of the Saddle

^{19/} Stickney, Michael and Bingler, E.C., "Earthquake Hazard Evaluation of Helena Valley Area, Montana," Open File Report 83. Montana Bureau of Mines and Geology, Butte, Montana, August 1981.

^{20/} Morrison-Maierle, Inc. "Chessman Reservoir and Events of October 1935" File Report by P. Porrini October 28, 1983, Helena, Montana.

Dam. The purpose of the test pits at the Main Dam was to expose the concrete core wall, verify its location and elevation, and visually examine its condition. Additionally, in-place field density tests were performed in the embankment. Core samples of the concrete cutoff were obtained, and the corewall was surveyed with a steel reinforcing bar locator. Test pits were also made on the southeast abutment of the Saddle Dam, which was not accessible with truck-mounted equipment.

The second phase of the field investigations consisted of drilling test borings at the Main Dam, and on the northwest abutment of the Saddle Dam. The purpose of these test borings was to determine the subsurface profile, in-place density, obtain disturbed and undisturbed samples for laboratory testing, and install piezometers at various depths. Additionally, relatively continuous core samples of the bedrock materials were obtained during the drilling program. Slotted pipe piezometers, surrounded with a filter fabric and gravel, and sealed with bentonite, were installed at the locations shown on Figure 11. All of the test pits and test borings were logged by a geologist or engineer from Northern Engineering and Testing.

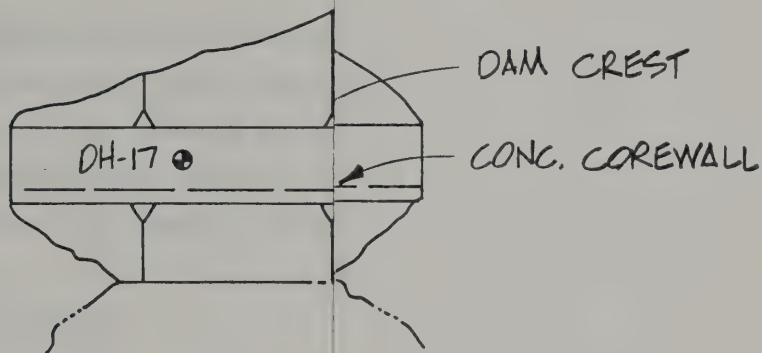
Five test pits and fifteen test borings were made at the locations shown on Figure 11. Boring and test pit depths ranged from about 5 to 79 feet. Elevations and locations of the borings and test pits were determined by Morrison-Maierle personnel. Boring and test pit locations are summarized as follows:

Drill Hole or
Test Pit Number

Approximate Location

TP-1	Station 0 + 92 on crest of Main Dam.
TP-2	Station 1 + 97 on crest of Main Dam.
TP-3	Station 2 + 87 on crest of Main Dam.
TP-4	Southeast abutment of Saddle Dam.
TP-5	Southwest abutment of Saddle Dam.
DH-6	Station 0 + 87 on crest of Main Dam downstream of concrete dam.
DH-7	Station 0 + 87 on crest of Main Dam upstream of concrete cutoff.
DH-8	Station 1 + 68 on crest of Main Dam upstream of concrete cutoff.
DH-9	Station 1 + 68 on crest of Main Dam downstream of concrete cutoff.
DH-10	Station 2 + 80 on crest of Main Dam downstream of concrete cutoff.
DH-11	Station 2 + 80 on crest of Main Dam upstream of cutoff.
DH-12	Station 1 + 43 on toe berm of Main Dam.
DH-13	Station 1 + 94 on toe berm of Main Dam.
DH-14	Station 2 + 46 on toe berm of Main Dam.
DH-15	Northeast abutment of Saddle Dam.
DH-16	On crest of Saddle Dam north of spillway.
DH-17	Station 3 + 92 on crest of Main Dam downstream of concrete cutoff.
DH-18	Station 1 + 17 on downstream slope, Main Dam.
DH-19	Station 1 + 80 on downstream slope, Main Dam.
DH-20	Station 2 + 65 on downstream slope, Main Dam.

APPROXIMATE EXTENT OF LIMITS OF EMBANKMENT
OF LOOSE SAND LAYER



LEGEND

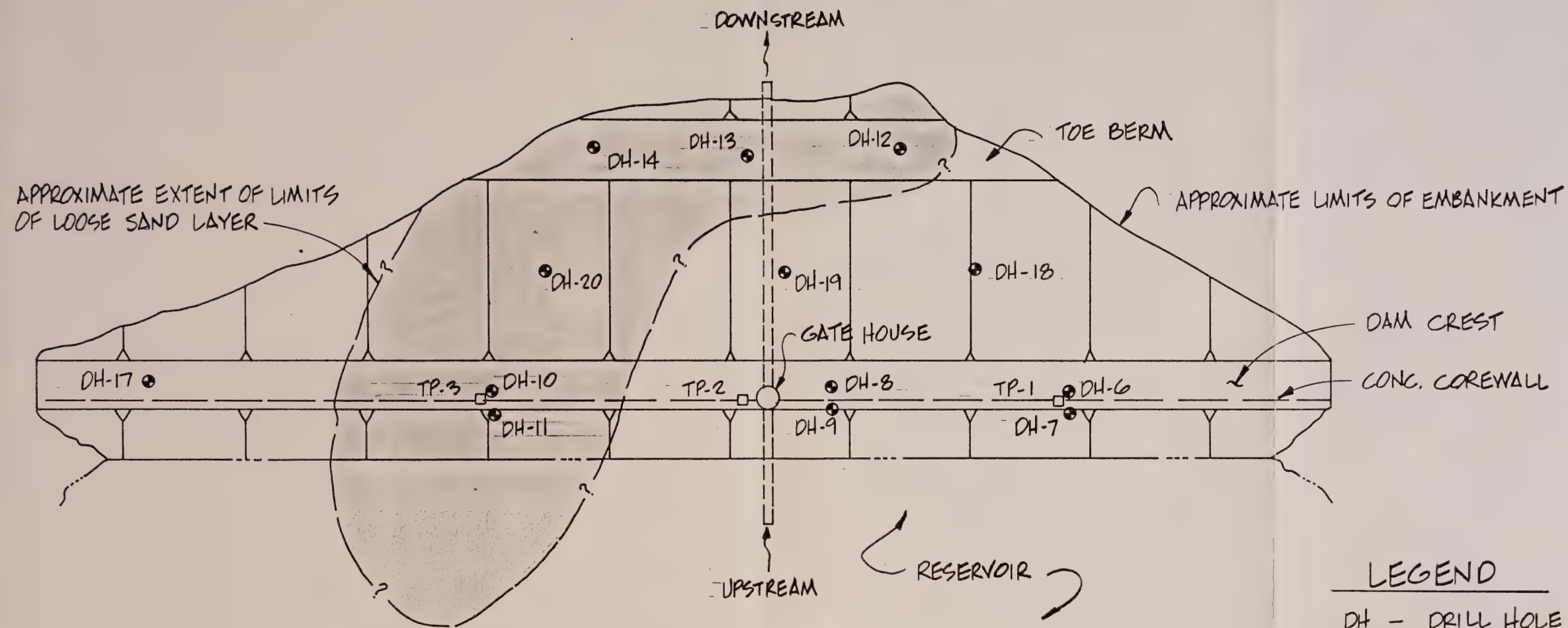
DH - DRILL HOLE
TP - TEST PIT

CHESSMAN DAM REHABILITATION INVESTIGATION LOCATION OF DRILL HOLES

FIGURE 11



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SCALE: 1" = 40'

LEGEND

DH - DRILL HOLE
TP - TEST PIT

CHESSMAN DAM
REHABILITATION INVESTIGATION
LOCATION OF DRILL HOLES
FIGURE 11

The third phase of the field investigations consisted of monitoring the piezometers installed in the drill holes. Initial drilling was completed in the first week of July 1983, and the piezometers were monitored by City of Helena personnel on approximately a weekly basis until the winter of 1983 and subsequent to that have been monitored intermittently until the reservoir was emptied during the summer of 1984. Drill Holes 18, 19 and 20 were drilled in mid-October 1983 after meetings with personnel from Morrison-Maierle and the City of Helena indicated the need for additional data. Piezometers are located at each drill hole shown on Figure 11.

3.4.2 Laboratory Investigations

Samples obtained during the field exploration were taken to the laboratory where they were carefully inspected and visually classified in accordance with the Unified Soils Classification System. Representative samples were selected for tests to determine engineering and physical properties of the soils, in general accordance with ASTM or other approved procedures.

<u>These included:</u>	<u>To determine:</u>
Grain Size Distribution	size and distribution of soil particles, i.e., clay, silt, sand, gravel.
Atterberg Limits.	the consistency and stickiness, as well as the range of moisture content within which the material is workable.
Natural Moisture.	moisture content representative of field conditions at time sample was taken.
Natural Density	dry unit weight of sample representative of in situ undisturbed condition.
Direct Shear.	soil shearing strength under varying load and/or moisture conditions. For use in foundation design and stability evaluation.
Triaxial Shear.	shear strength parameters of soils under carefully controlled conditions. Used for stability and dam evaluation.

These included

To determine

Moisture Density Relationship . . .the optimum (best) moisture content for compacting soil and the maximum dry unit weight (density) for a given compactive effort.

In addition, a rock quality designation number (RQD) was determined for each core run by summing the length of all sound, hard pieces of core over 4 inches in length, and dividing this number by the total length of the core run. This value, along with the core recovery percentage, is recorded on the test boring logs.

3.4.3 Area Description, Geology and Subsurface Profile

The site is located in the Helena National Forest. The terrain is generally mountainous and forested.

The predominant geologic feature is the Boulder Batholith, which is a large igneous intrusive formation, consisting primarily of quartz monzonite and granodiorite. Occasional alaskite intrusion in the quartz monzonite are present. This area is seismically active.

The subsurface profile at the Main Dam consists of silty or clayey sand fill, or sandy clay fill overlying alluvial deposits of sand, clay, or gravel, and then quartz monzonite bedrock. The depth of the fill in the drill holes ranges from a maximum of about 42 feet near the center of the dam, to about 6 feet at the toe. The thickness of the alluvial deposits below the embankment range from more than 40 feet in Drill Hole 10 on the south side of the dam, to about 3 feet in Drill Hole 17 on the south abutment.

At the Saddle Dam, clayey sand fill was encountered near the spillway to a depth of 14 feet where in-place clayey sand is present. Below the sand, quartz monzonite was present at a depth of about 20 feet. On the abutments at the Saddle Dam, quartz monzonite is usually present within several feet of the ground surface. These materials are described in detail on the test pit and boring logs, and are discussed below:

Fill - The embankment fill at both dams appears to consist mostly of decomposed quartz monzonite. Most of the fill consists of clayey sand, although there are occasional, apparently non-continuous zones of silty sand and sandy clay fill. Standard penetration resistance (N) values in the fill ranged from 2 to 23 blows per foot, and averaged about 11. Field and laboratory tests are summarized below:

In-place dry density, from field density tests, pcf.93 to 106

Maximum dry density, ASTM D698, pcf.117

Angle of internal friction, from direct shear and
consolidated undrained triaxial test, degrees.28 to 40

Cohesion intercept, from direct shear and
consolidated undrained triaxial tests, psf0 to -1700

Silty Sand - Silty sand was encountered below the Main Dam embankment, primarily on the south side of the dam, and beneath the toe. It appears that there is a continuous layer of very loose silty sand at the following locations:

<u>Drill Hole</u>	<u>Approximate Depth, feet</u>	<u>Approximate Elevation</u>
DH-11	33 to 43	61 to 71
DH-12	6 to 12	64 to 70
DH-13	10 to 14	61 to 65
DH-14	10 to 13	62 to 65
DH-15	20 to 27	63 to 70

The location of this layer is shown on Figures 11 and 12. N values in these layers ranged from about 2 to 25 and averaged about 10.

Clayey Sand and Clay - The clayey sand and silty and sandy clay below the embankment are generally loose to firm and have moderate strength parameters. The results of laboratory tests on clay materials are summarized below:

In-place dry density, pcf.89 to 94

Angle of internal friction from consolidated undrained
triaxial tests, degrees.18 to 26

Cohesion intercept from consolidated undrained triaxial
tests, psf300

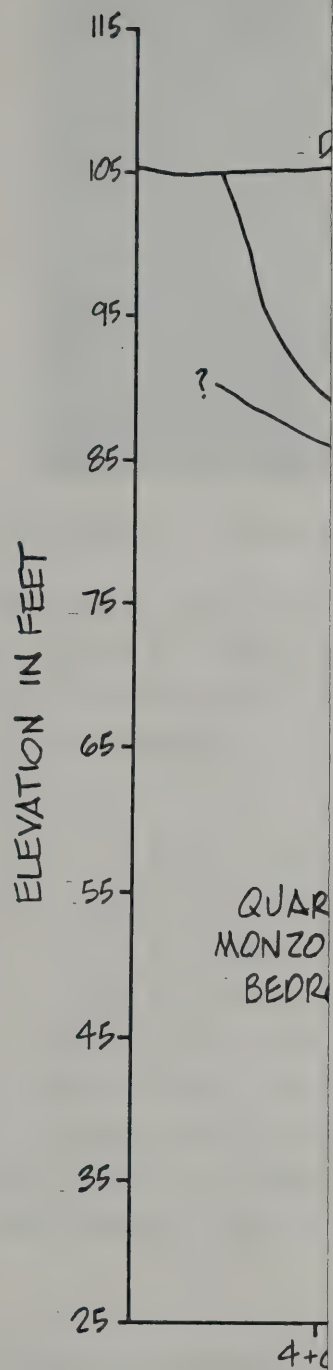
Gravel and Boulders - Deposits of sandy gravel with cobbles and boulders are present at the Main Dam below the fine grained clays and sands. N values in these materials are high, normally more than 50.

Quartz Monzonite Bedrock - The quartz monzonite bedrock is weathered near the contract, but becomes very competent with several feet of penetration. It is fractured and jointed, and contains occasional alaskite prophyry dikes.

The concrete cutoff wall was closely inspected in Test Pits 1 through 3. It was encountered at about elevation 101 and is approximately 1 foot wide at the top. The condition of the cutoff appeared to be good. Cracking was not evident and no reinforcing bars were detected or observed in the wall. Since borings made immediately adjacent to the cutoff revealed in-place sand, clay and gravel between the embankment fill and the quartz monzonite bedrock, it can be concluded that the cutoff does not extend to bedrock. The exposed cutoff, as well as concrete cores obtained from the wall, are shown below. Further information on the concrete cores is summarized in Northern Engineering and Testing's December 1, 1983 report.



Photograph 1 - Concrete cutoff wall exposed in Test Pit 3.

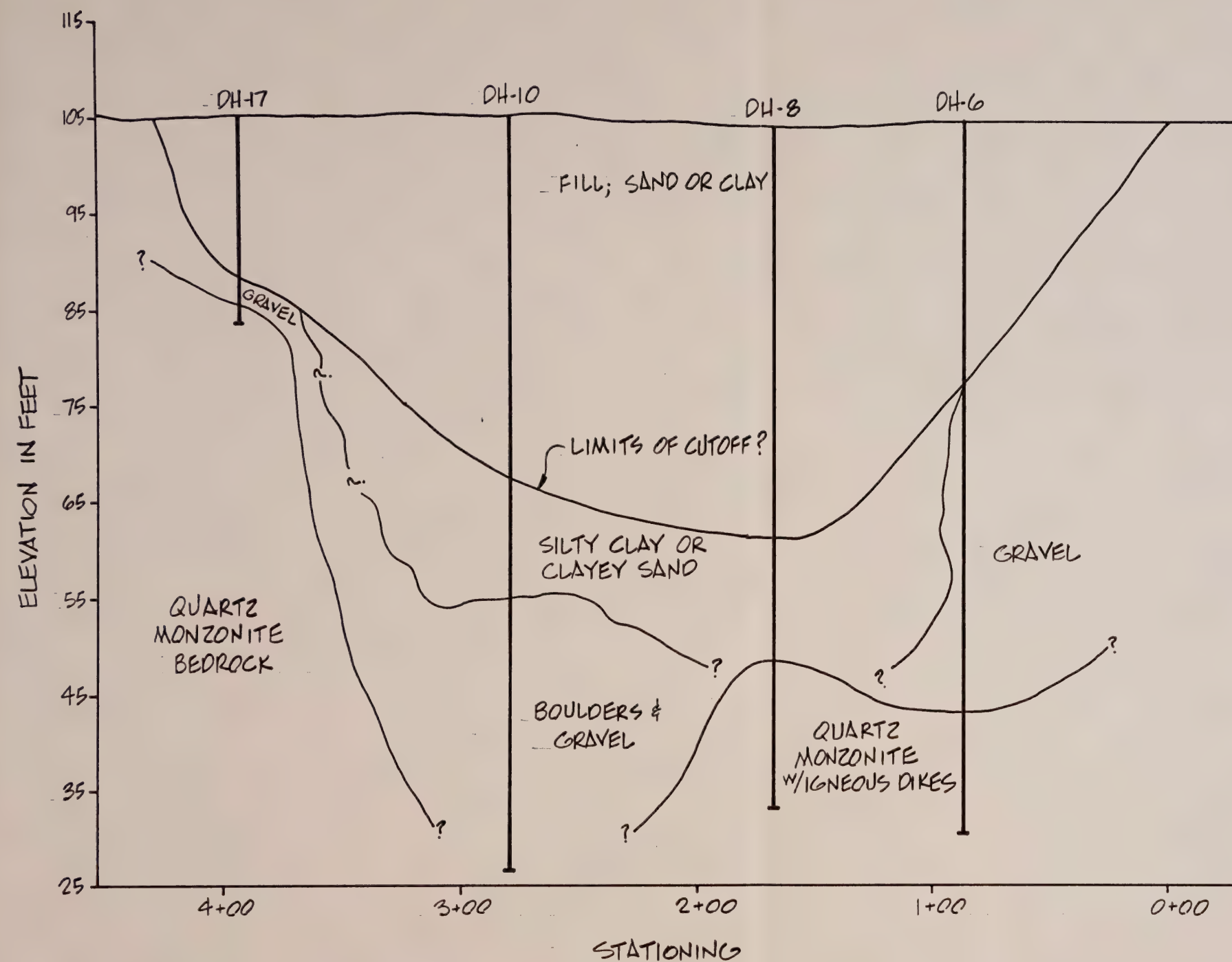


**CHESMAN DAM
REHABILITATION INVESTIGATION
TYPICAL GEOLOGIC SECTION
ALONG DAM AXIS**

FIGURE 12



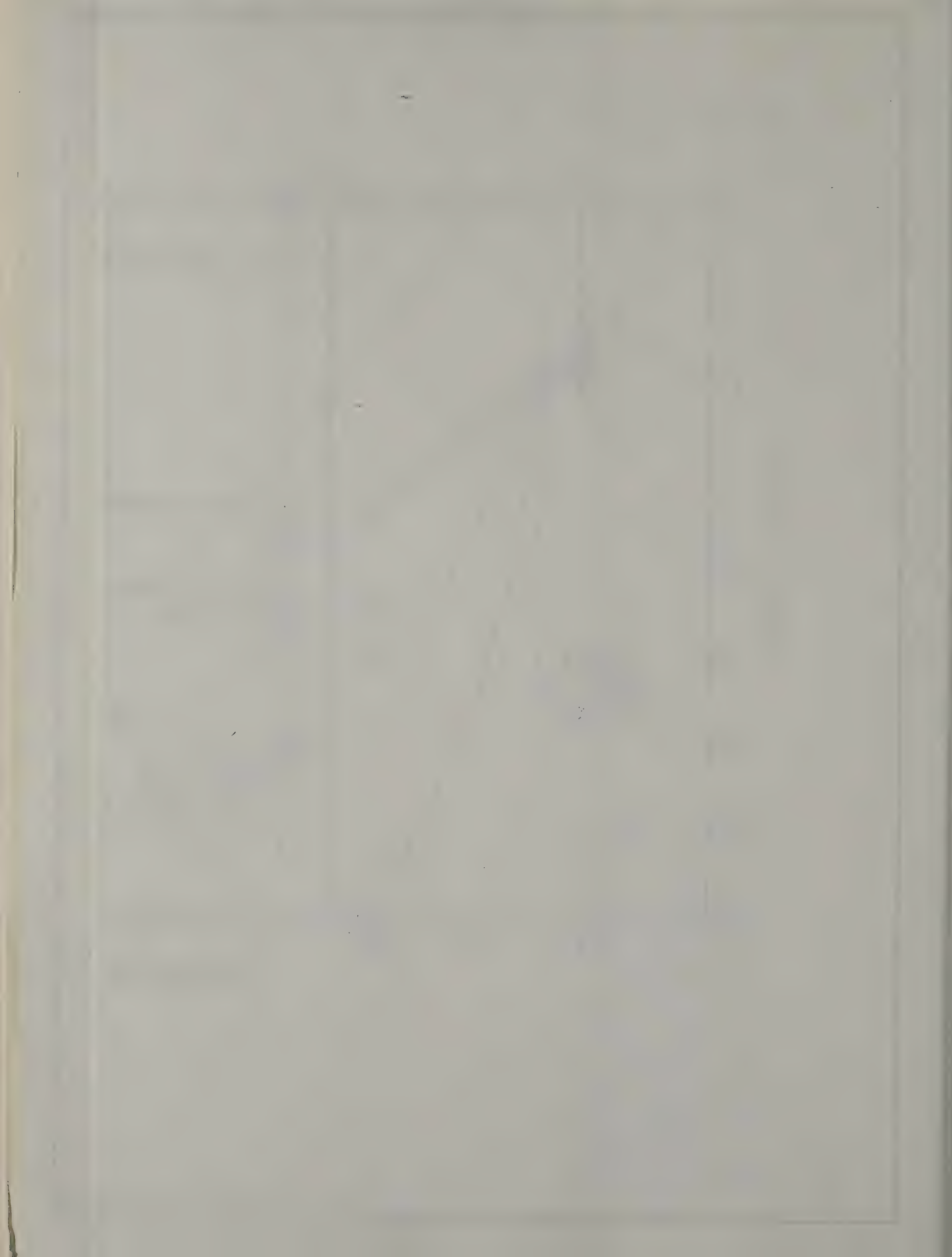
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CHESMAN DAM
REHABILITATION INVESTIGATION
TYPICAL GEOLOGIC SECTION
ALONG DAM AXIS

FIGURE 12







Photograph 2 - Concrete cores obtained from cutoff at Test Pit 1.

Piezometers were installed at various locations in the drill holes as shown on Figure 11. These piezometers have been monitored as the reservoir was lowered since early summer 1983. The results of this monitoring are shown in the APPENDIX.

Generally, water levels upstream of the concrete cutoff correspond to reservoir levels, within several feet. Immediately downstream of the cutoff, piezometer levels are lower than the corresponding upstream piezometers, in some instances by 5 to 10 feet. Piezometers installed in the natural material below the cutoff, indicate higher levels than those installed downstream of the cutoff. Excess hydrostatic pressure is present in the foundation soil at the toe of the dam. This was indicated by artesian flow from the borings at the time of drilling and piezometer levels which were several feet above the ground surface when the reservoir was high.

3.4.4 Stability Analysis - Methods and Design Criteria

Evaluation of the stability of an earth dam can generally be divided into two areas of consideration: the stability under static conditions, and the stability under seismic or earthquake loading. The stability under static conditions was analyzed by a circular arc, method of slices analysis. This method requires identification of the subsurface profile and strength parameters for each different material. The critical embankment



THE HISTORY OF THE UNITED STATES

The history of the United States is a story of growth and change. It begins with the first settlers who came to the shores of the New World. They brought with them the traditions and customs of their homelands. As time passed, these settlers adapted to their new environment and created a new society. The story of the United States is a story of the struggle for freedom and the pursuit of the American dream.

The early years of the United States were marked by conflict and uncertainty. The colonies fought for their independence from Britain. They won the American Revolution and established a new nation. The new nation was faced with many challenges. It had to build a government and a system of laws. It had to defend itself against foreign threats. It had to unite the people and create a sense of national identity. The story of the United States is a story of the triumph of the human spirit over adversity.

THE AMERICAN REVOLUTION

The American Revolution was a turning point in the history of the United States. It was a time of great change and growth. The colonies fought for their independence from Britain. They won the American Revolution and established a new nation. The new nation was faced with many challenges. It had to build a government and a system of laws. It had to defend itself against foreign threats. It had to unite the people and create a sense of national identity. The story of the United States is a story of the triumph of the human spirit over adversity.

section (usually the highest), is then analyzed by assuming circular failure surfaces. For each surface, the forces tending to cause failure (gravity and seepage), and the forces tending to resist failure (friction and cohesion), are calculated. By assuming different failure circle locations, the surface with the minimum factor of safety is located. Normally, the downstream slope is analyzed for steady state seepage, and the upstream slope is analyzed for sudden drawdown from maximum reservoir level.

Two methods were used in analyzing slopes for stability under earthquake loading. A conventional pseudo-static method of analysis was used to evaluate the effect of earthquake forces on slope stability. This method is similar to the method described above, except a horizontal force is applied to the slope to simulate the earthquake forces. The other method used concerned the potential for liquefaction in the embankment materials and foundation soils.

In selecting the strength parameters for stability analysis, and the minimum acceptable factors of safety for each reservoir loading condition, the Corps of Engineers have established recommended guidelines ^{21/}. For given reservoir conditions the minimum factors of safety and strength parameters to be used are summarized as follows:

Case	Loading Condition	Minimum Factor of Safety	Shear Strength Parameters to Use
I	Sudden drawdown from spillway crest	1.2	Minimum composite of drained and undrained strength parameters.
II	Partial pool with horizontal steady state seepage	1.5	Average of drained and undrained parameters for undrained less than drained - Use drained for undrained greater than drained.

^{21/} Department of the Army, Corps of Engineers, "Recommended Guidelines for Safety Inspection of Dams," Washington, D.C., 1972.

Case	Loading Condition	Minimum Factor of Safety	Shear Strength Parameters to Use
III	Steady state seepage	1.5	Same as Case II
IV	Earthquake - Cases II and III with seismic loading	1.0	Same as Case II or III

In evaluating the strength data of the Chessman Main Dams, the consolidated, undrained triaxial tests with pore pressure measurement were considered as the undrained tests, and the direct shear tests were considered as the drained tests. These test results are summarized as follows:

Summary of Direct Shear - Drained Tests

Drill Hole	Depth (feet)	Material	Friction Angle ϕ (degrees)	Cohesion Intercept $\bar{C}$ (psf)
6	14.3 - 14.5	FILL, Sand	40	0
9	9.2 - 9.8	FILL, Sand	30	1700
9	19.3 - 19.5	FILL, Sand	28	950

Summary of Consolidated Undrained Triaxial Tests - Undrained Tests

8	33.5 - 43.5	FILL, Sand	39	0
8	43.5 - 45.5	CLAY, Silty	26	0
13	18.0 - 20.0	CLAY, Silty	18	300

For Case I, evaluation of the above data for the embankment fill indicates a friction angle of 28 degrees and a cohesion intercept of 0 was used. For the foundation soil, a friction angle of 18 degrees and a cohesion intercept of 300 psf was used.

For Cases II through IV, for the fill, a friction angle of 28 degrees and a cohesion intercept of zero was used. For the foundation soil, a friction angle of 26 degrees and a cohesion intercept of 300 psf was used. These values are slightly lower than strict adherence to the guidelines would indicate but are considered reasonable for the material types and



conditons which exist and contribute to instability. These conditions include 1) high artesian pressures in the foundation and embankment, 2) layering in the embankment soils, and 3) the non-homogeneous nature of the foundation.

The potential for liquification of the embankment or foundation soil must also be evaluated under earthquake conditions. Experience has shown that in certain loose sand soils below the water table, earthquake forces can cause pore water pressures to build up. When the pore water pressure reaches the same level as the overburden stress, the deposit loses its strength and assumes a liquid consistency. If a continuous layer of material, extending under an embankment liquifies during an earthquake, failure of the embankment can occur. In order to liquify, a sand deposit must classify as a clean or silty sand, have a loose consistency, and occur below the water table. Determination of the susceptibility of a deposit to liquefaction, involves tests of the physical properties of samples to determine their classification. If a deposit has been identified as a clean or silty sand, methods have been developed using the standard penetration test, to determine if the deposit is loose enough to liquify. ^{22/}

In order to check seismic stability, it is necessary to select a design earthquake. For both the pseudo-static analysis and the liquefaction analysis, it is necessary to use a maximum estimated ground acceleration. A literature search was made to determine past earthquake history in the area, and select a design ground acceleration.

3.4.5 Results of Stability Analysis

In the geotechnical analysis, the material properties and design parameters indicated for Cases I through II, listed previously, were used in determining the Factors of Safety at reservoir elevations 96 and 99 feet.

^{22/} Seed, H.B., Idriss, I.M., Arango, I., "Evaluation of Liquefaction Potential Using Field Performance Data," Journal of Geotechnical Engineering, American Society of Civil Engineers, Vol. 109, No. 3, March, 1983, pp. 458-482.

A maximum ground acceleration of 0.15 times gravity was used for the pseudo-static seismic evaluation of the embankment section at Station 1 + 68. The subsurface profile with the maximum thickness of low strength alluvial deposits was used. This appeared to occur in Drill Hole 8. Water levels within the embankment were determined from the piezometer monitoring, with the reservoir at elevation 99. The results of the slope stability analyses for the existing Main Dam embankment were as follows:

<u>Reservoir Loading Condition</u>	<u>Calculated Minumum Apparent Factor of Safety</u>		<u>Recommended Factor of Safety</u>
	<u>Reservoir Elevation 96</u>	<u>Reservoir Elevation 99</u>	
Full Reservoir, steady state seepage, without earthquake	1.3	1.1	1.5
Full Reservoir, steady state seepage, with earthquake	0.8	0.7	1.0
Sudden drawdown from full reservoir	1.0	1.0	1.2

By following procedures for evaluating the liquefaction potential, the sand zone identified on Figure 11 was found to be liquefiable at accelerations in the range of 0.05 g to 0.07 g. The Montana Bureau of Mines publication Memoir 51 - Montana Earthquakes 1869 to 1979 indicates a maximum horizontal acceleration of 0.118 g was recorded in Helena during the Magnitude 6.0, October 31, 1935 earthquake. This publication also indicates a Magnitude 6.0 earthquake in the Helena-Ovando region has a recurrence interval of 70 years, based on statistical analysis of past events. Another Bureau publication Open File Report 83-Earthquake Hazard Evaluation of the Helena Valley Area, Montana also indicates that simultaneous rupture along all fault segments of the Helena Valley fault zone would produce an earthquake in the magnitude 7.0 to 7.5 range. For these reasons, it was concluded there was a high possibility of liquefaction and additional analysis to determine the probability was not conducted.

A seismic coefficient of 0.15 was used in the pseudostatic earthquake analysis. This value was selected after consultation with Dr. H. Bolton Seed, a seismic expert at the University of California (Dr. Seed's review of the Chessman field data is included in the APPENDIX). The seismic coefficient is slightly higher than the 0.1 value suggested by the Corps of Engineers for a Zone 3 earthquake area. It should be noted that the seismic coefficient used in the pseudostatic analysis is not the same as the maximum ground acceleration which can be expected at a site. The 0.15 value also reflects judgement after detailed studies at other nearby sites with similar geologic settings. ^{23/}

3.4.6 Rehabilitation Alternatives for the Main Dam Embankment

In order to upgrade the Main Dam embankment to meet current safety standards, two areas will require modification. First, it will be necessary to densify at least a portion of the loose sand layer below the south portion of the dam, to eliminate the possibility of liquefaction. Second, the stability of the embankment will need to be improved to meet recommended minimums.

Several methods may be used for densifying the loose sand layer. These include excavating the layer and recompacting it, or by compaction grouting. Overexcavating and recompacting the loose sand is feasible near the toe, but would require removing the existing embankment and dewatering. This is not feasible under the higher portions of the dam.

Compaction grouting would involve injecting a low slump soil cement into the loose sand layer. Compaction grouting is different from chemical grouting or the normal grouting performed to limit water flow through a soil or rock. The solid cement grout remains as a distinct homogeneous mass during injection. Densification is achieved through the process of

^{23/} "Site Seismicity Evaluation, Mystic Lake Dam Project", International Engineering Company, August 1983.

displacement. A verification program would be incorporated into the grouting program to determine that the loose layer had actually been densified. Compaction grouting has successfully been used during the last 20 years and methods have been developed for the design and control of the grouting program. Depending upon the alternate selected, additional test borings would be performed prior to finalizing the grout program, to better define the limits of the loose sand layer.

Compaction piling was also considered. With this system, displacement piling such as timber piles would be driven through the loose layer. Compaction would be achieved by displacement and vibration. In order to densify the sand properly, it appears it would be necessary to drive piling on about 2 to 4 foot centers. This would result in an excessive number of piling and cost many times more than compaction grouting.

In order to increase the factors of safety for slope stability, several modifications will be required:

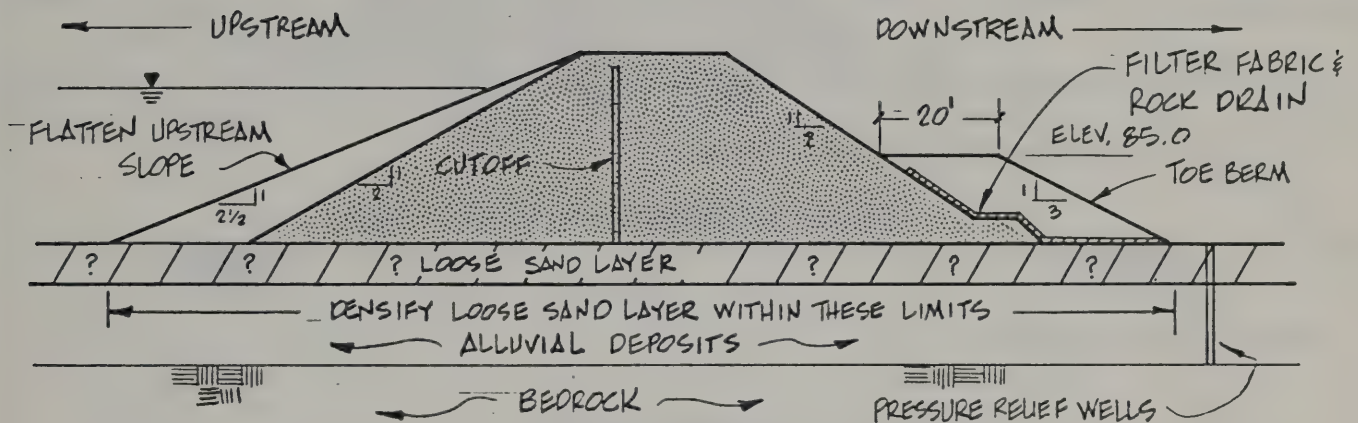
- 1) flattening the upstream slope to about $2\frac{1}{2}H:1V$;
- 2) flattening the downstream slope to about $3H:1V$, or constructing a toe berm; and
- 3) installing pressure relief wells in the foundation soils at the toe of the dam to relieve hydrostatic pressure.

A granular filter, or a filter fabric and rock drain will be necessary between the downstream embankment and toe berm, or between the existing embankment and flattened slope. Alternates were considered for densifying the loose sand zone, and increasing the slope stability. Four alternates are presented ranging from total densification by compaction grouting to removal and replacement of loose sand material.



Alternate 1

Alternate 1 involves densifying all of the loose sand layer below the embankment and construction of a 20 foot wide toe berm to elevation 85 on the toe.

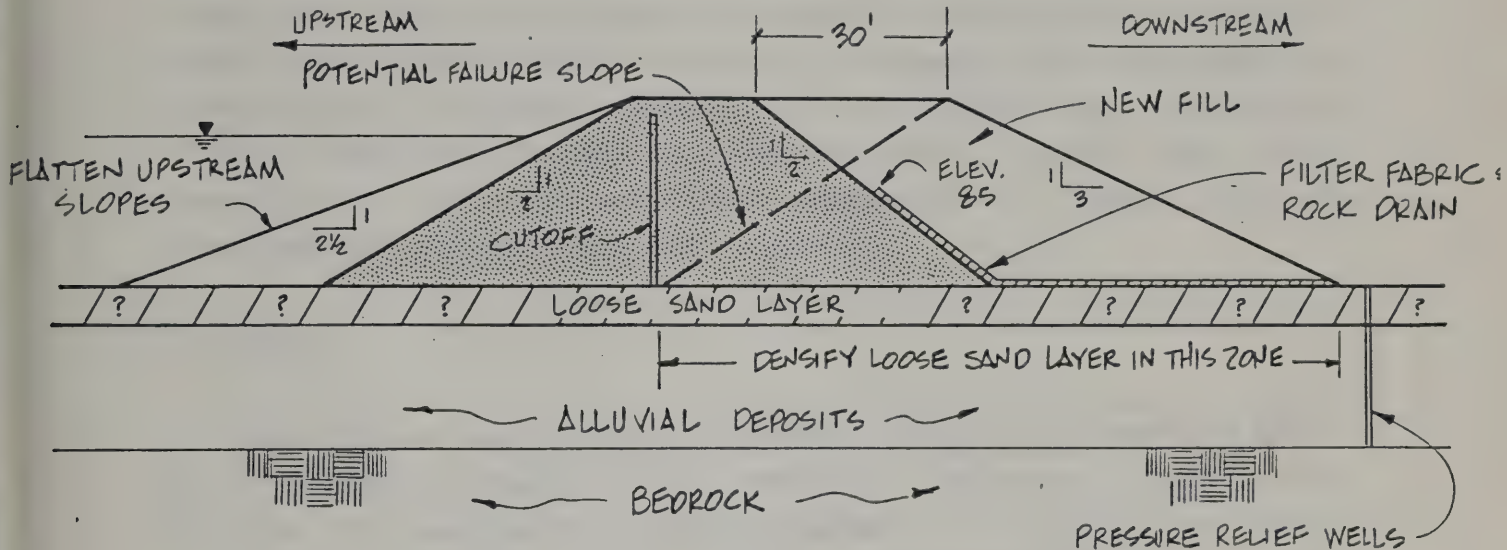


ALTERNATE 1 (N.T.S.)

With this alternate, the entire loose sand layer would be densified by compaction grouting. It would also be necessary to conduct additional test borings upstream of the crest, after the reservoir was drawn down, to define the upstream limits of the loose sand. The toe berm, drain, pressure relief wells and upstream slope flattening would extend across the entire dam. With this slope configuration, factors of safety of 1.6 and 1.0 result for the static and seismic steady state seepage cases respectively. For the sudden reservoir drawdown, the factor of safety is about 1.2.

Alternate 2

Alternate 2 involves densifying the loose sand layer downstream of the corewall, widening the crest by 30 feet, and flattening the downstream slope to 3H:1V.



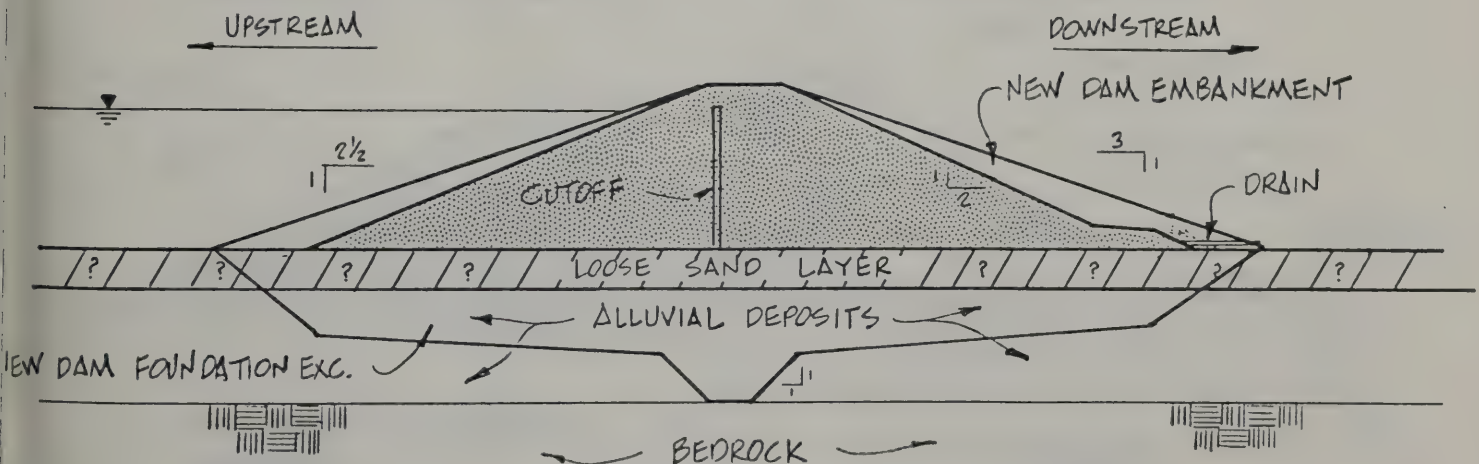
ALTERNATE 2 (N.T.S.)

An advantage of this alternate is that only a portion of the loose sand zone would be densified. By densifying the sand downstream of the corewall, and maintaining fill within a zone inclined at 1H:1V from the base of the corewall, an additional factor of safety may be realized. Even if the upstream sand liquifies and the slope fails, there should be sufficient embankment remaining to prevent loss of the reservoir. Since densification of the sand is a major expense item, this alternate is more economical. Where the loose sand zone is not present, it would not be necessary to increase the crest width by the 30 feet, but the downstream slopes should be flattened to 3H:1V and the other stabilization measures provided. The drain, pressure relief wells and upstream slope flattening would extend across the entire dam. Minimum factors of safety for this configuration were calculated as 1.5 and 1.0 for the static and seismic steady state seepage cases, respectively. For the sudden reservoir drawdown, the factor of safety is approximately 1.2.

Construction cost estimates for Main Dam rehabilitation of Alternates 1 and 2 are nearly equal at \$504,640 and \$487,970, respectively. These costs include compaction grouting, embankment modifications, core wall and outlet extensions and do not consider Saddle Dam spillway modifications, contingencies, mobilization and engineering. For Alternate 1, the upstream loose sand layer and corresponding compaction grouting limits were estimated to the extent shown on Figure 11 and require additional subsurface investigation to accurately define the boundaries and costs. Alternate 2 involves less compaction grouting than Alternate 1, however, these savings are offset by more subexcavation and embankment fill. Overall there are less uncertainties associated with Alternate 2.

Alternate 3

An alternate was developed to eliminate the need for compaction grouting. Alternate 3 considered the removal and replacement of the existing Main Dam. By removing the existing embankment and excavating the foundation soils to bedrock, the stability of the replacement structure would be improved. A new structure would have a broader base than the existing Main Dam resulting from flatter upstream and downstream slopes and would include a cutoff wall, outlet works and embankment drains. The construction cost estimate for a new dam (Alternate 3) is \$1,164,000. This cost does not include Saddle Dam spillway modifications, contingencies, mobilization and engineering.

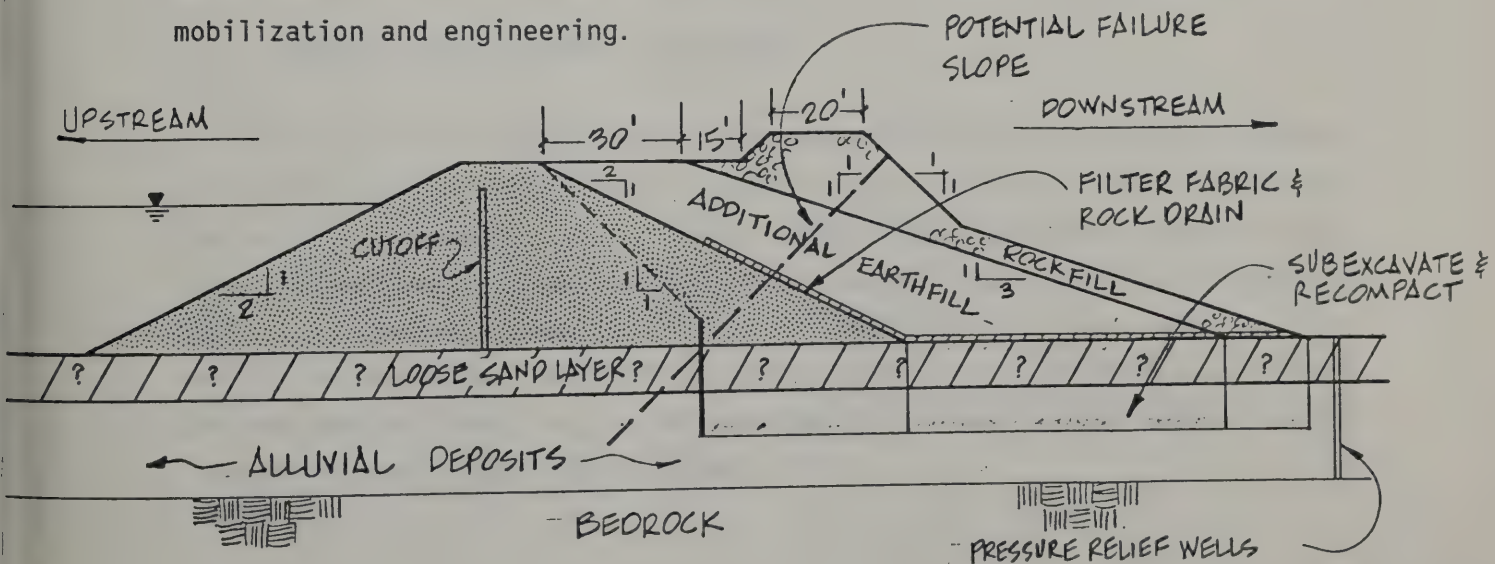


ALTERNATE 3 (N.T.S.)

Alternate 4

An additional alternate was examined to restore the operation of the project while increasing the stability of the Main Dam. The fourth alternate allows for complete use of the reservoir to elevation 99 feet and would protect against the sudden failure of the Main Dam. Alternate 4 provides additional material to be placed on the downstream slope of the Main Dam and eliminates the upstream slope flattening, compaction grouting and extension of the core wall. Rock material on the downstream slope would provide stability during upstream slope failures or dam crest overtopping situations. The rockfill would also provide protection against instantaneous failure by preventing the total washout of the embankment.

Alternate 4 assumes greater owner risk than with the other alternates presented in this report, as minimum factors of safety for embankment stability will not likely be met by this plan. Optimum dimensions have not been designed for the additional materials, however, an estimate has been prepared using the assumptions shown on the following sketch. In addition to placing a large volume of earth and rock on the downstream face of the Main Dam on a 3H to 1V slope, drains and pressure relief wells are included to improve stability. Extension of the outlet works is also included in the cost estimate for Alternate 4 which totals \$382,100. This cost estimate does not include modifications to the Saddle Dam, contingencies, mobilization and engineering.



ALTERNATE 4 (N.T.S.)

SUMMARY OF COST ESTIMATES FOR REHABILITATION OF THE MAIN DAM

Alternate 1 - Compaction grouting	\$ 504,640*
Alternate 2 - Partial grouting ds berm	487,970*
Alternate 3 - Replacement embankment	1,164,000*
Alternate 4 - Ds berms rockfill	382,100*

- * The above estimates do not include necessary modifications to the Saddle Dam, contingencies, mobilization and engineering and provide only a comparison of costs for the range of alternate rehabilitation plans presented herein.

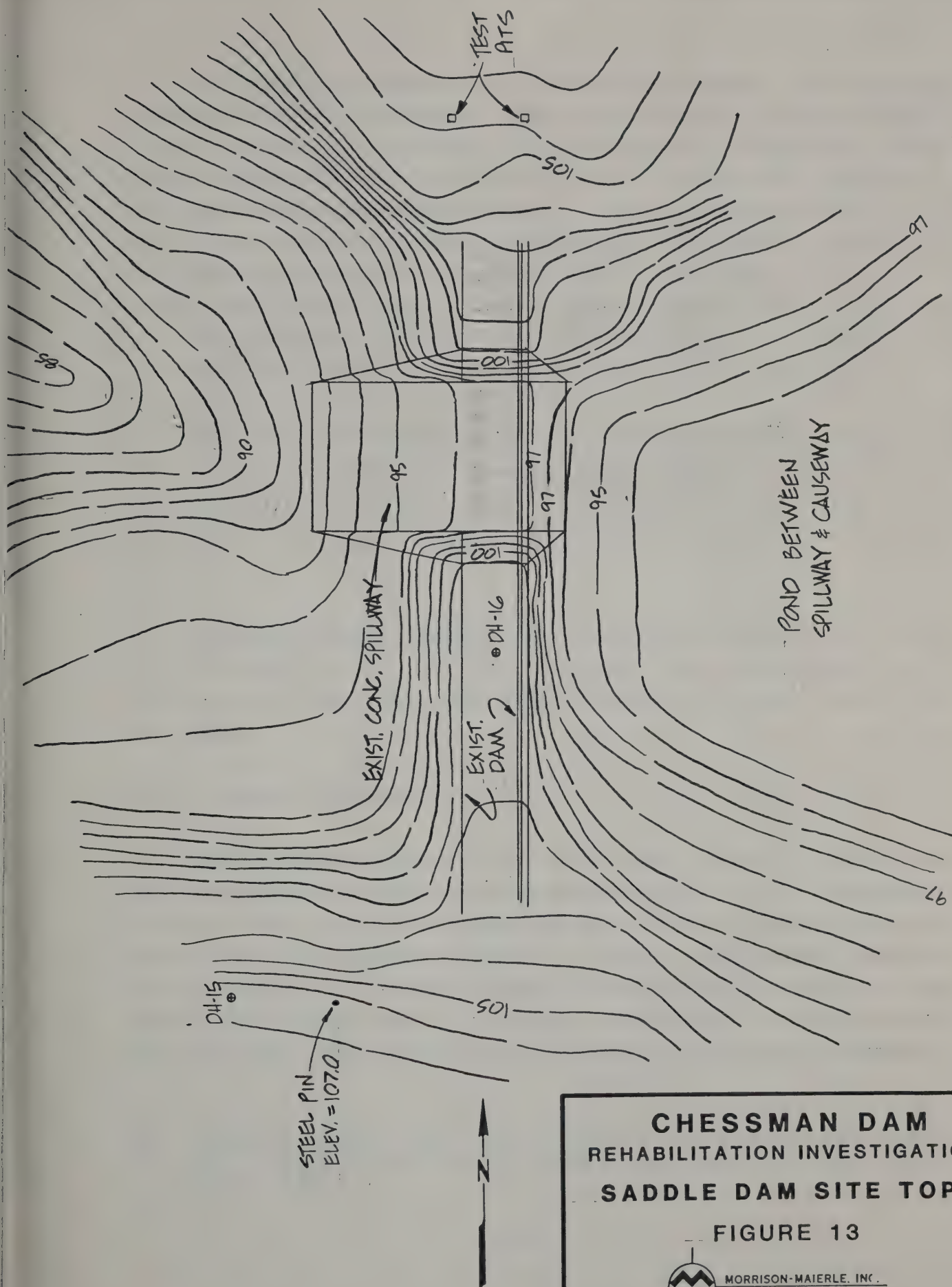
3.5 PRELIMINARY ENGINEERING DESIGN AT SADDLE DAM

3.5.1 Topographic Survey

A topographic survey at the Saddle Dam site was conducted on August 9 and 10, 1983. A level loop originating from project datum at the Main Dam was extended to two control points at the Saddle Dam. Results of this survey are shown on Figure 13. Additionally, cross sections of the discharge channel between the Forest Service causeway across the reservoir and the proposed location of a new Forest Service roadway 300 feet downstream of the spillway were acquired.

3.5.2 Forest Service Roadway (Causeway)

The existing Forest Service roadway provides access between the Park Lake Road and the town of Rimini and must remain open to traffic. However, the existing roadway cuts off the Saddle Dam embankment from the main reservoir by forming a causeway and does not allow the spillway to function until the reservoir reaches a stage of 102 feet or 1 foot over the core wall in the Main Dam. Because the outlet works are of limited capacity the Chessman Project must rely on the emergency spillway at the Saddle Dam for release of significant flood inflows.



**CHESMAN DAM
REHABILITATION INVESTIGATION
SADDLE DAM SITE TOPO**

FIGURE 13



MORRISON-MAIERLE, INC.
CONSULTING ENGINEERS

The existing causeway poses a dangerous situation. The following example indicates the potential danger created by this spillway obstruction: The reservoir is normally filled during the spring runoff period. During the period May 21-22, 1981 basins such as Chessman were subjected to extreme rainfall and moderate snowmelt. Records at Frohner Meadow, which is in the vicinity of Chessman, indicate that 4.4 inches of rainfall and 1.8 inches of snowmelt contributed to runoff. The reservoir was not full at the start of this event. However, had the reservoir been at elevation 99 feet, the 1981 storm could have increased storage to elevation 103. This would have threatened the stability of the Main Dam embankment.

Discussion with the Helena National Forest indicate that Forest Service road number 299 which crosses Chessman Reservoir is scheduled for relocation in the Spring of 1985. ^{24/} Alignment of the proposed roadway will cross the spillway discharge channel 300 feet downstream of the Saddle Dam.

Backwater analysis indicates that the proposed roadway will not influence discharges at the spillway. Therefore, when the causeway road is removed and the new roadway constructed discharges at the spillway will not be impeded.

3.5.3 Hydraulic Considerations

Routing studies of the spillway design flood (Storm B) indicate that the spillway structure must discharge approximately 2,000 cfs. Topographic features limit the existing Saddle Dam and spillway to 105 feet of crest width before extensive bedrock removal is required. Furthermore, operation of the project to elevation 99 would leave two feet of operating head between the spillway and the existing corewall crest in the Main Dam. Discharge under these constraints would require a coefficient of discharge

^{24/} Telephone conversations: Morrison-Maierle, Inc. and U.S. Forest Service - Helena National Forest, August 8, 1983, March 27, 1984, September 28, 1984.

value equal to 6.7, an impossibility. Therefore, a greater operating head is required which can be accomplished by lowering the spillway crest or extending the corewall elevation in the main dam. Both alternatives were considered in this report.

Spillway Operating Alternate A

- ° Crest width 105 feet at elevation 97 feet
- ° Loss of 200 acre-feet or 65 million gallons from storage at elevation 99.

Spillway Operating Alternate B

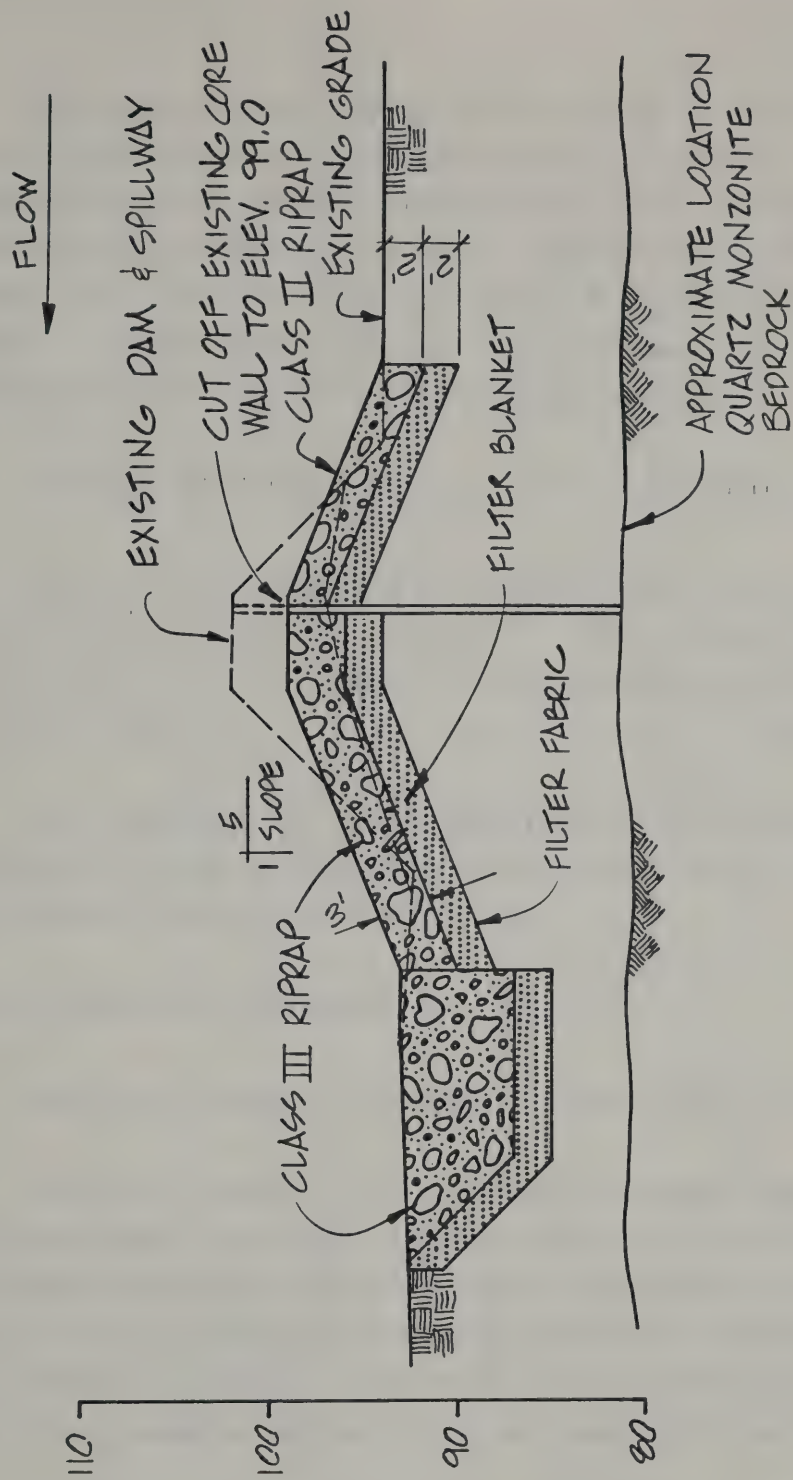
- ° Crest width 105 feet at elevation 99 feet
- ° Restoration of storage to maximum normal pool
- ° Requires extension of corewall in Main Dam

Two types of spillway structures would meet design criteria at a reasonable economic cost: modification of the existing structure with riprap protection and the Soil Conservation Service (SCS) Type B concrete drop spillway.

Spillway Structure Alternate 1 - Modification of Existing Structure

Incorporation of the existing structure would provide the least costly solution. The proposed spillway structure described herein is considered an emergency structure to be used on an infrequent basis and not for routine operation of the project.

It is proposed for either spillway operating alternative that the Saddle Dam corewall be cut off at the respective crest elevation and protection of the downstream face be added to allow the overtopping of the Saddle Dam. A typical section is shown on Figure 14. The proposed structure basically constructed of rock with a concrete corewall would act as a broad crested weir. Rating curves to determine discharge capacity were developed using a backwater analysis which started 300 feet downstream and proceeded into the reservoir.



SCALE: 1" = 20' HORIZ.
1" = 10' VERT.

CHESSMAN DAM **REHABILITATION INVESTIGATION** **SPILLWAY STRUCTURE ALT. 1**

FIGURE 14



MORRISON-MAIERLE, INC.
 CONSULTING ENGINEERS

Overtopping during the SDF could last up to three days, with the major peak occurring within a 24 hour period. Therefore, the face protection must withstand erosive and piping action for a considerable period. A well graded dumped riprap protection is proposed with a nominal thickness of 36 inches and underlain by a well graded filter blanket and a filter fabric liner. Some rehabilitation of this structure will be required following the occurrence of an extreme meteorological event.

Spillway Structure Alternate 2 - SCS Type B Drop Spillway

Hydraulic proportioning of the SCS Type B Drop Spillway was developed using the SCS Engineering Handbook.^{25/} The structural height and discharge requirements of this project could be handled by a Type B configuration as shown on Figure 15.

This shape is the least costly concrete structure and would perform adequately during discharge of a SDF event. Minor scour might occur just downstream of the stilling basin lip.

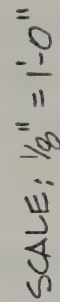
3.5.4 Structural Considerations

Spillway Structure Alternate 1 - Modification of Existing Structure

Stability analysis of the existing Saddle Dam indicates that the existing slopes should be flattened from 2H:1V to 2½ H:1V. Slopes on the riprapped protected structure would be flattened to 5H:1V (see Figure 14). Original construction plans and the drill hole investigation indicate that the Saddle Dam corewall appears to extend to the quartz monzonite bedrock. Therefore, seepage and uplift do not appear to be significant problems.

A two foot extension of the corewall will be required at the location of the existing spillway to bring it up to elevation 99 feet.

^{25/} U.S. Soil Conservation Service, Engineering Handbook Drop Spillway, Section 11, Washington, D.C., 1968



MORRISON-MAIERLE, INC.
CONSULTING ENGINEERS

Spillway Structure Alternate 2 - SCS Type B Drop

The Type B Drop Spillway structure would tie to the existing corewall. This will limit cutoff wall requirements to four feet to meet stability requirements and give an adequate factor of safety for seepage and uplift considerations.

It is suspected that over a 50 year life a concrete structure would suffer from weathering action and frost heave.

3.5.5 Geotechnical Considerations

Two drill holes and two test pits were used to investigate the Saddle Dam site. The first drill hole (DH-15) was located approximately 40 feet downstream of the Saddle Dam corewall in the left abutment. Quartz monzonite bedrock was encountered at elevation 104.0 feet or a depth of 3.0 feet.

The second drill hole (DH-16) was located just downstream of the Saddle Dam corewall and ten feet north of the spillway sidewall. Quartz monzonite bedrock was encountered at elevation 82.5 feet or a depth of 19.5 feet. Materials encountered in the Saddle Dam embankment were decomposed quartz monzonite and clayey sand with scattered layers of silty sand and gravels. Blow counts ranged from 7 to 9 blows per foot. Unified Soils Classification of the Saddle Dam material was SC.

Two test pits were excavated in the right abutment (TP-4 & 5) approximately 30 feet south of the spillway sidewall and 15 feet upstream and downstream of the Saddle Dam corewall. Bedrock at these locations was encountered between elevations 102.2 and 100.5 feet.

These bedrock locations agree well with the location shown on the original design drawings for the bottom of the corewall.



3.5.6 Preliminary Design Drawings

Preliminary design drawings of the proposed spillway at the Saddle Dam are shown on Figures 16 and 17.

3.5.7 Cost Estimate

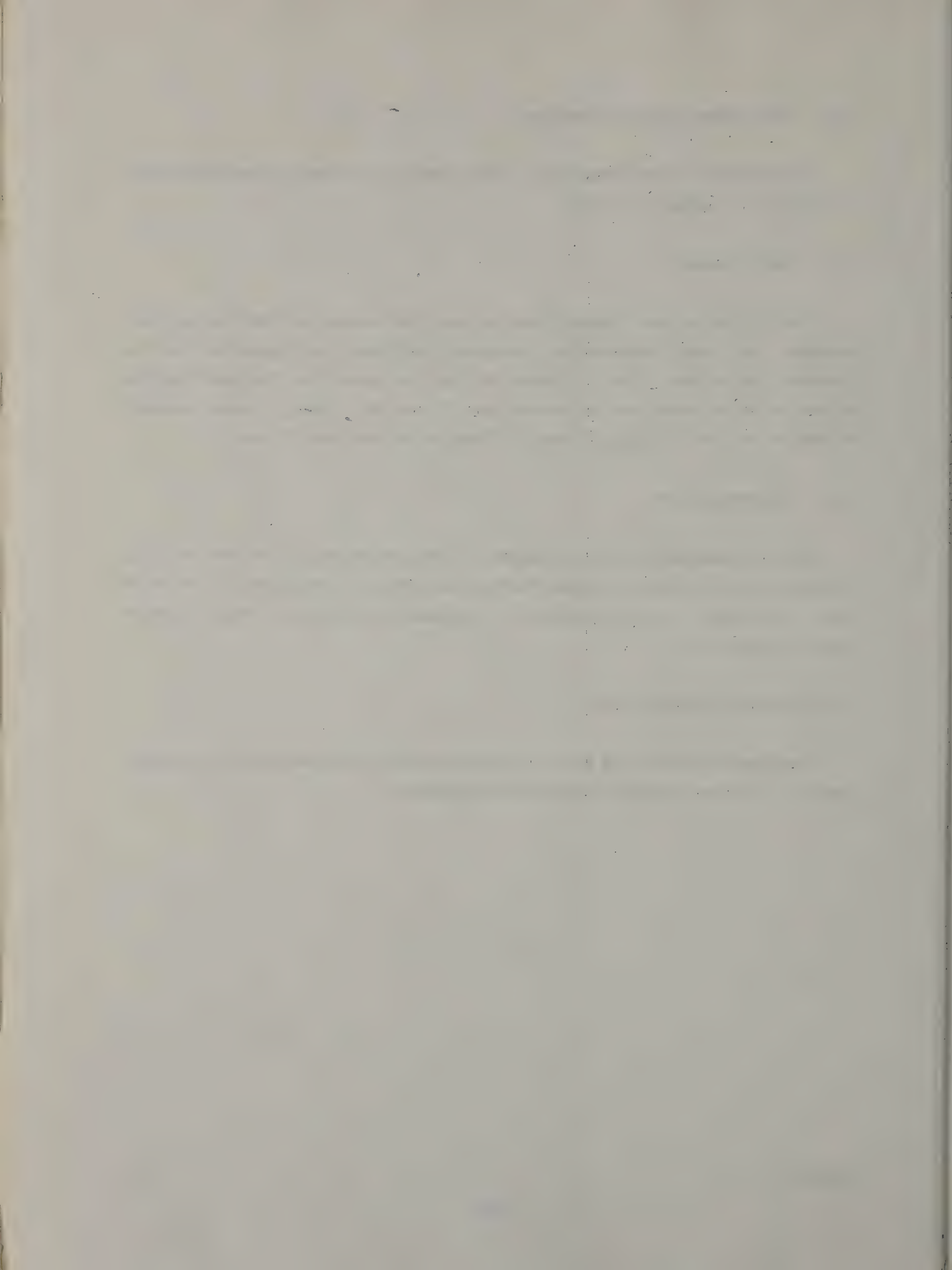
The following preliminary design cost estimates in 1984 dollars are provided for each alternative emergency spillway configuration at the Chessman Saddle Dam. Table 3 shows the cost estimates for the modification of the existing structure at elevations 97 and 99. Table 4 shows the cost estimates for the SCS Type B Drop Spillway at elevations 97 and 99.

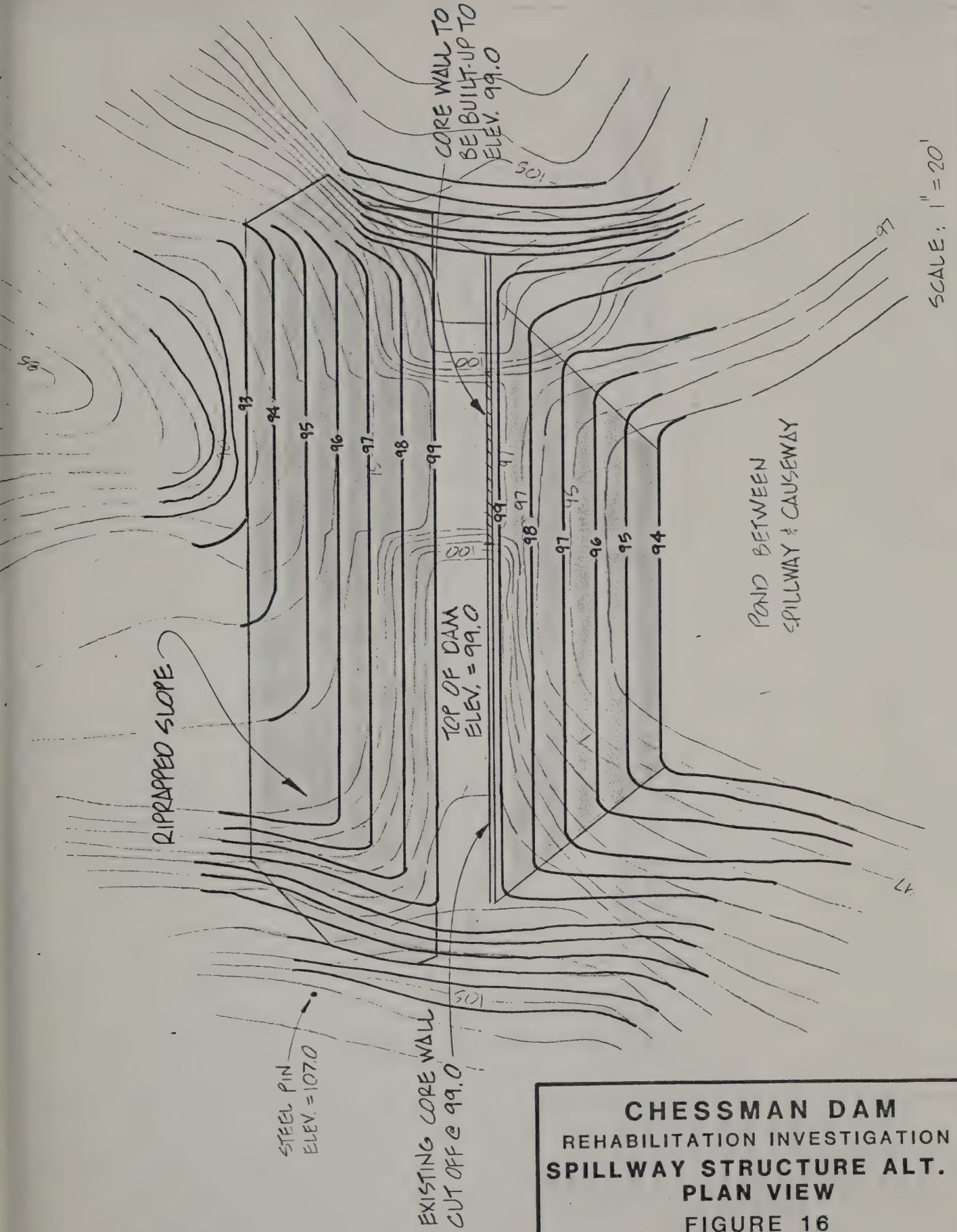
3.5.8 Recommendation

The recommended spillway design is the modification of the existing structure with riprap and establishing the core wall crest at elevation 99 feet. The cost for this portion is included in the total rehabilitation plan in Chapter 4.4.

3.6 EMERGENCY WARNING PLAN

The Emergency Warning Plan is contained in a separate report entitled: Part II - Chessman Rehabilitation Investigation.



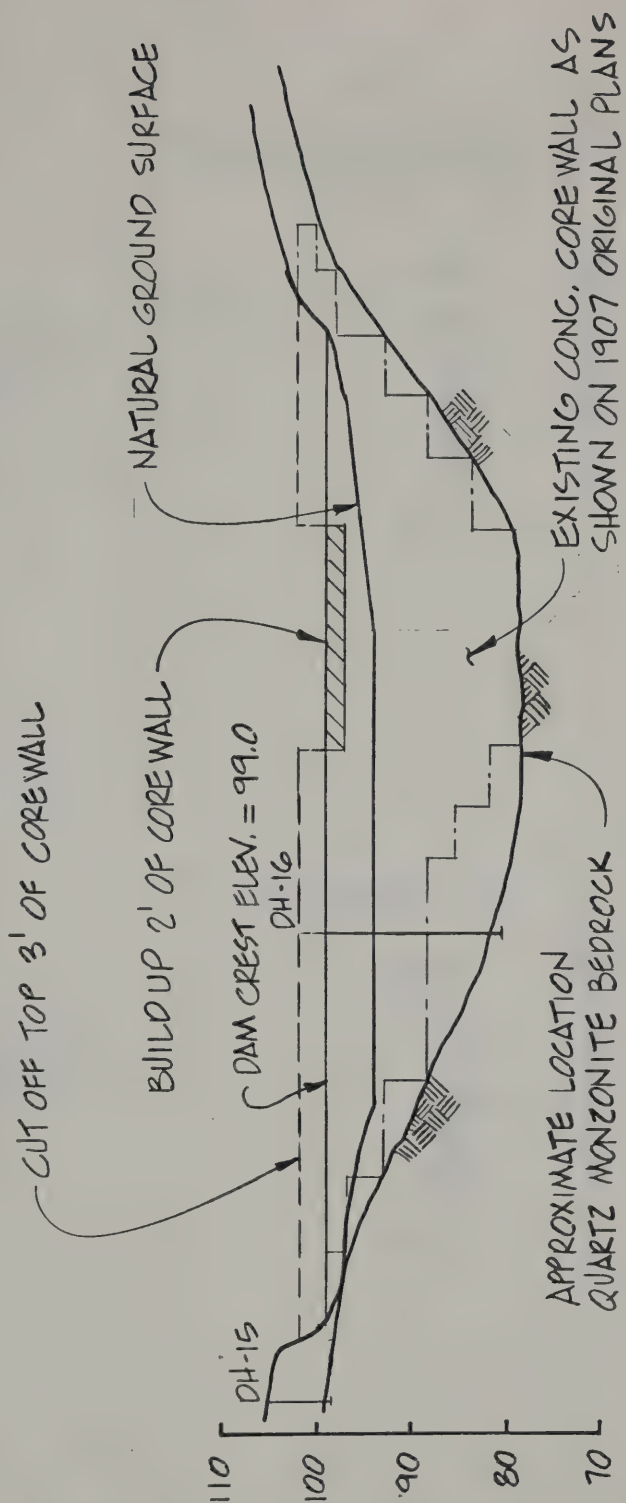


CHESSMAN DAM
 REHABILITATION INVESTIGATION
SPILLWAY STRUCTURE ALT. 1
PLAN VIEW
FIGURE 16



MORRISON-MAIERLE, INC.
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CHESSMAN DAM
REHABILITATION INVESTIGATION
SPILLWAY STRUCTURE ALT. 1
PROFILE
FIGURE 17



MORRISON-MAIERLE, INC.
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1875-1876

1876-1877

1877-1878

1878-1879

1879-1880

1880-1881

Table 3

SPILLWAY STRUCTURE ALTERNATE 1 - RIPRAP
CONSTRUCTION COST ESTIMATE
@ ELEVATION 97 FEET

<u>ITEM</u>	<u>DESCRIPTION</u>	<u>QUANTITY</u>	<u>UNIT</u>	<u>UNIT PRICE</u>	<u>AMOUNT</u>
1.	Earthwork	2,850	CY	\$ 4.00	\$11,400.00
2.	Corewall inspection	400	CY	6.00	2,400.00
3.	Remove corewall to 97	19	CY	100.00	1,900.00
4.	Remove spillway	42	CY	100.00	4,200.00
5.	Membrane lining	1,170	SY	2.00	2,340.00
6.	Filter blanket	780	CY	9.00	7,020.00
7.	Class 3 riprap	1,330	CY	25.00	33,250.00
8.	Class 2 riprap	115	CY	20.00	2,300.00
TOTAL					\$64,810.00

@ ELEVATION 99 FEET

<u>ITEM</u>	<u>DESCRIPTION</u>	<u>QUANTITY</u>	<u>UNIT</u>	<u>UNIT PRICE</u>	<u>AMOUNT</u>
1.	Earthwork	2,750	CY	\$ 4.00	\$11,000.00
2.	Corewall inspection	470	CY	6.00	2,820.00
3.	Remove corewall to 99	12	CY	100.00	1,200.00
4.	Remove spillway	42	CY	100.00	4,200.00
5.	Membrane lining	1,325	SY	2.00	2,650.00
6.	Filter blanket	985	CY	9.00	8,865.00
7.	Class 3 riprap	1,480	CY	25.00	37,000.00
8.	Class 2 riprap	315	CY	20.00	6,300.00
TOTAL					\$74,035.00

TABLE 4

SPILLWAY STRUCTURE ALTERNATE 2 - CONCRETE DROP
CONSTRUCTION COST ESTIMATE

@ ELEVATION 97 FEET

<u>ITEM</u>	<u>DESCRIPTION</u>	<u>QUANTITY</u>	<u>UNIT</u>	<u>UNIT PRICE</u>	<u>AMOUNT</u>
1.	Earthwork - remove existing dam	670	CY	\$ 4.00	\$ 2,680.00
2.	Remove corewall and spillway	108	CY	100.00	10,800.00
3.	Remove and stockpile riprap	30	CY	9.00	270.00
4.	Place reinforced concrete spillway	170	CY	400.00	68,000.00
5.	Furnish and place riprap key	360	CY	25.00	9,000.00
TOTAL					\$90,750.00

@ ELEVATION 99 FEET

<u>ITEM</u>	<u>DESCRIPTION</u>	<u>QUANTITY</u>	<u>UNIT</u>	<u>UNIT PRICE</u>	<u>AMOUNT</u>
1.	Earthwork - remove existing dam	670	CY	\$ 4.00	\$ 2,680.00
2.	Remove corewall and spillway	108	CY	100.00	10,800.00
3.	Remove and stockpile riprap	30	CY	9.00	270.00
4.	Place reinforced concrete spillway	260	CY	400.00	104,000.00
5.	Furnish and place riprap key	360	CY	25.00	9,000.00
TOTAL					\$126,750.00

4.0 RECOMMENDATIONS FOR PROJECT REHABILITATION

4.1. PROJECT OPERATION

The evaluation of the Chessman Project has determined that operation of the project at a maximum storage elevation of 99 feet would take best advantage of existing facilities and would optimize the operation of this water supply. The requirements of project operation at elevation 99 include:

- Reconstruction of the spillway at the Saddle Dam
- Extension of the corewall in the Main Dam embankment
- Stabilization of the Main Dam embankment by compaction grouting, additional material and drains

Operation of the project could be limited to elevation 97 feet, however, cost savings at the Saddle Dam and Main Dam are offset by loss of 200 acre-feet or 65,000,000 gallons of active storage. The economic conclusion reached is that the cost per gallon of additional rehabilitation to operate at elevation 99 is less than the overall rehabilitation cost per gallon, \$0.0014 versus \$0.0016, therefore, relatively comparable in the rehabilitation plan of restoring full operation of the project. Increases in storage above elevation 99 feet were given consideration, however, cost increases are much greater as additional work is required on the Main Dam embankment, core wall and outlet extensions. Limitations of inflow capacity and water rights indicate the potential availability of additional water to be marginal. Filling an extra 200 to 500 acre-feet would take approximately 8 - 20 days longer each season.

Awareness of the project's present condition resulted in the City's choice to drain the Chessman reservoir in the summer of 1984. This recommended operational status is an interim measure until the project is rehabilitated and/or addressed in the ongoing water supply and treatment evaluation for the City of Helena.

4.2 MAIN DAM

The geotechnical evaluation concluded that the stability of the Main Dam embankment is marginal under static conditions and if an earthquake occurs there could be failure due to slope instability, or liquefaction of the foundation soils. The factors of safety for stability of the Main Dam embankment do not satisfy recommended minimums.

To upgrade the Main Dam embankment to meet federal dam safety guidelines, modifications are required. The layer of loose sand in the foundation along the southern portion of the Main Dam must be densified to eliminate the liquefaction potential. Compaction grouting was considered the most applicable method to densify the loose sand and can be accomplished by a specialty contractor.

To increase the stability of the Main Dam under static conditions requires several modifications which include flattening both the upstream and downstream slopes and installing drains and pressure relief wells in the foundation soils at the toe to relieve hydrostatic pressure.

Four alternatives were presented in Chapter 3.4.6 to upgrade the Main Dam embankment:

Alternate 1: densifies all loose sand in and below the embankment and increases the cross section dimension of the dam by adding a toe berm on the downstream face and flattening the upstream slope.

Alternate 2: densifies the loose sand layer in and below the embankment downstream of the corewall and widens the crest width by 30 feet on the downstream side and flattens the upstream slope.

Alternate 3: removes and replaces the Main Dam with an embankment founded on bedrock and with flatter slopes for improved stability.

Alternate 4: Densifies the loose sand layer downstream of the cutoff wall and increases downstream embankment by widening the crest width by 30 feet and adding rockfill to the downstream face to protect against overtopping or sudden embankment failure.

To meet the objective of this rehabilitation investigation by restoring the operation of the project at the current engineering design standards, Alternate 2 is recommended as it meets the minimum factors of safety recommended in the dam safety guidelines. Since the limits of the loose sand layer upstream of the core wall could not be assessed, there is a degree of uncertainty in estimating the Alternate 1 costs. Furthermore, removal and replacement of the Main Dam is more costly than either compaction grouting alternate. Alternate 4 which costs less than Alternate 2 was not recommended as it assumes greater owner risk since minimum factors of safety are not met. It would require additional evaluation of design criteria, risk and acceptance level from all involved concerns.

In addition to densification and embankment modifications for stability, extension of the corewall to the Main Dam's crest (elevation 105) is recommended. Extension of the corewall will require excavation of the top five foot of dam height to expose the existing corewall. Steel ties would be provided between the old and new material and a waterproof sealant applied to the upstream face of the joint. The dam crest would be replaced and compacted. The extended corewall will provide for operational latitude and routing of the spillway design flood.

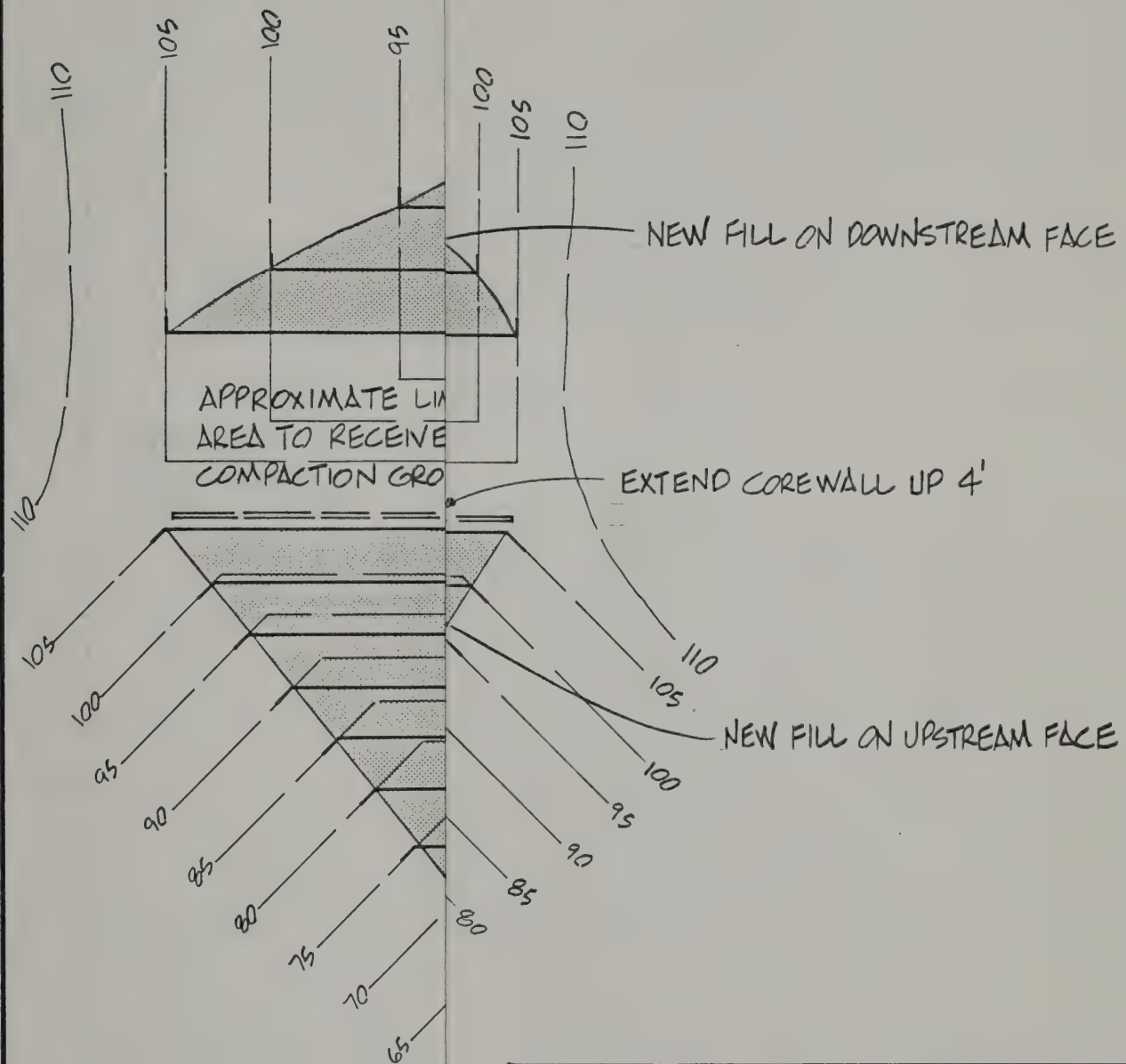
Replacement of the outlet works originally recommended in the draft report has been eliminated from consideration. An inspection of the outlet by City staff on August 16, 1984, indicates that the visual condition of the cast iron pipe is adequate, while the concrete conduit requires only minimal repairs after 80 years of service. Detailed examination of the gate valve could not be made, but visible gate seals looked fine. The television log of the inspection is attached in the APPENDIX. All costs for outlet replacement have been removed from the cost estimates in this report.

4.3 SADDLE DAM

Modification of the existing structure to elevation 99 feet and protecting all exposed slopes with riprap is recommended. The use of materials abundant in this area results in a less costly structure than the reinforced concrete drop spillway and is adequate for the infrequent use of the spillway facility.

The preliminary design drawing for the modifications are shown on Figures 16 and 17.

PRESSURED TO ELEV.
7 TYP. RECOMPACTED



CHESSMAN DAM
REHABILITATION INVESTIGATION

PLAN VIEW
MAIN DAM REHABILITATION

FIGURE 18



MORRISON-MAIERLE, INC.
CONSULTING ENGINEERS

PRESSURE RELIEF WELLS
7 TYP.

THIS AREA TO BE EXCAVATED TO ELEV.
45.0 & BACKFILLED & RECOMPACTED

NEW 24" RCP
EXT. W/OUTLET STRUCT.

NEW FILL ON DOWNSTREAM FACE

APPROXIMATE LIMITS OF
AREA TO RECEIVE
COMPACTION GROUTING

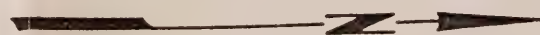
EXISTING OUTLET WORKS

EXTEND COREWALL UP 4'

NEW FILL ON UPSTREAM FACE

NEW 16" C.I. EXTENSION
W/TRASH RACK

RESERVOIR



SCALE: 1" = 40'

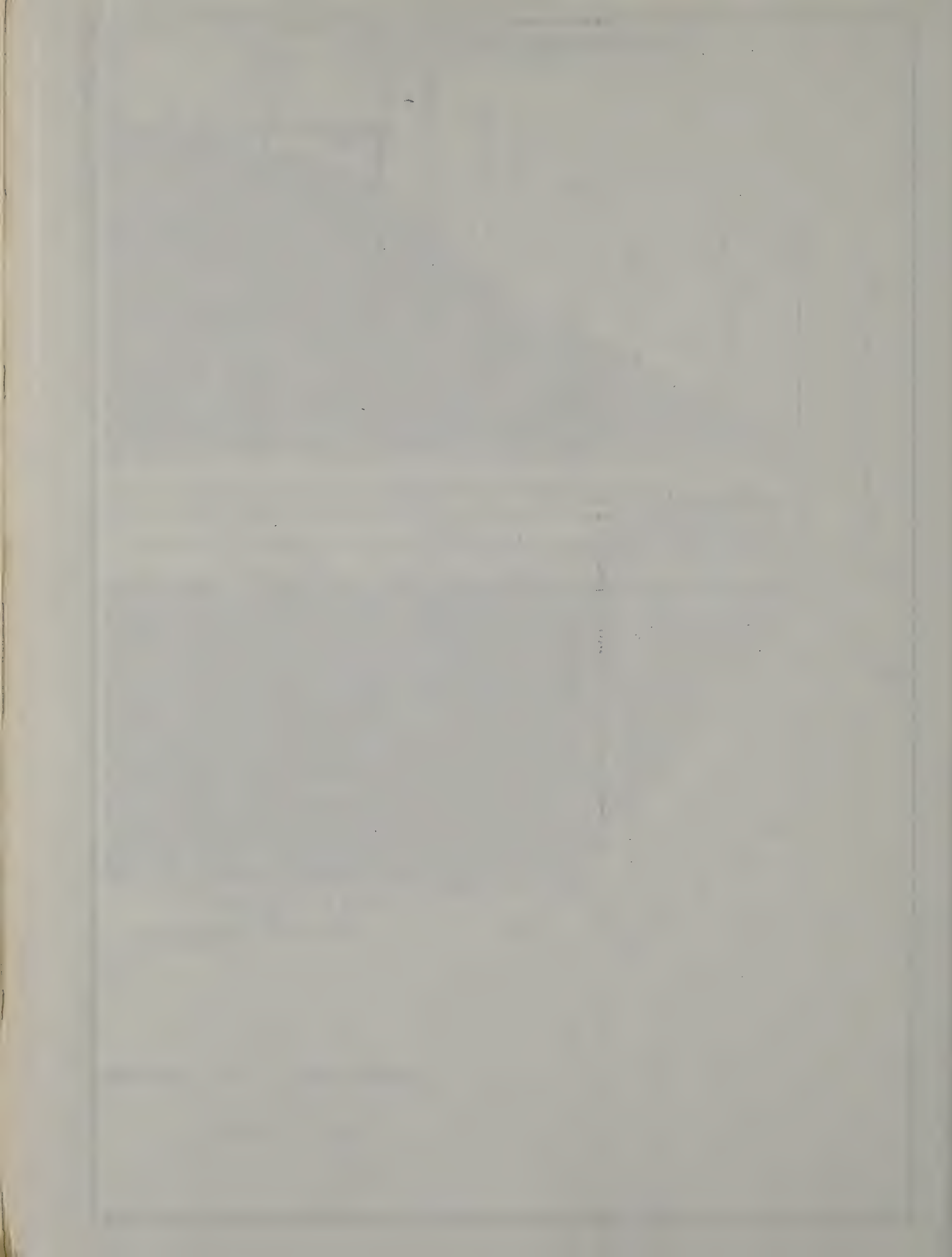
**CHESSMAN DAM
REHABILITATION INVESTIGATION**

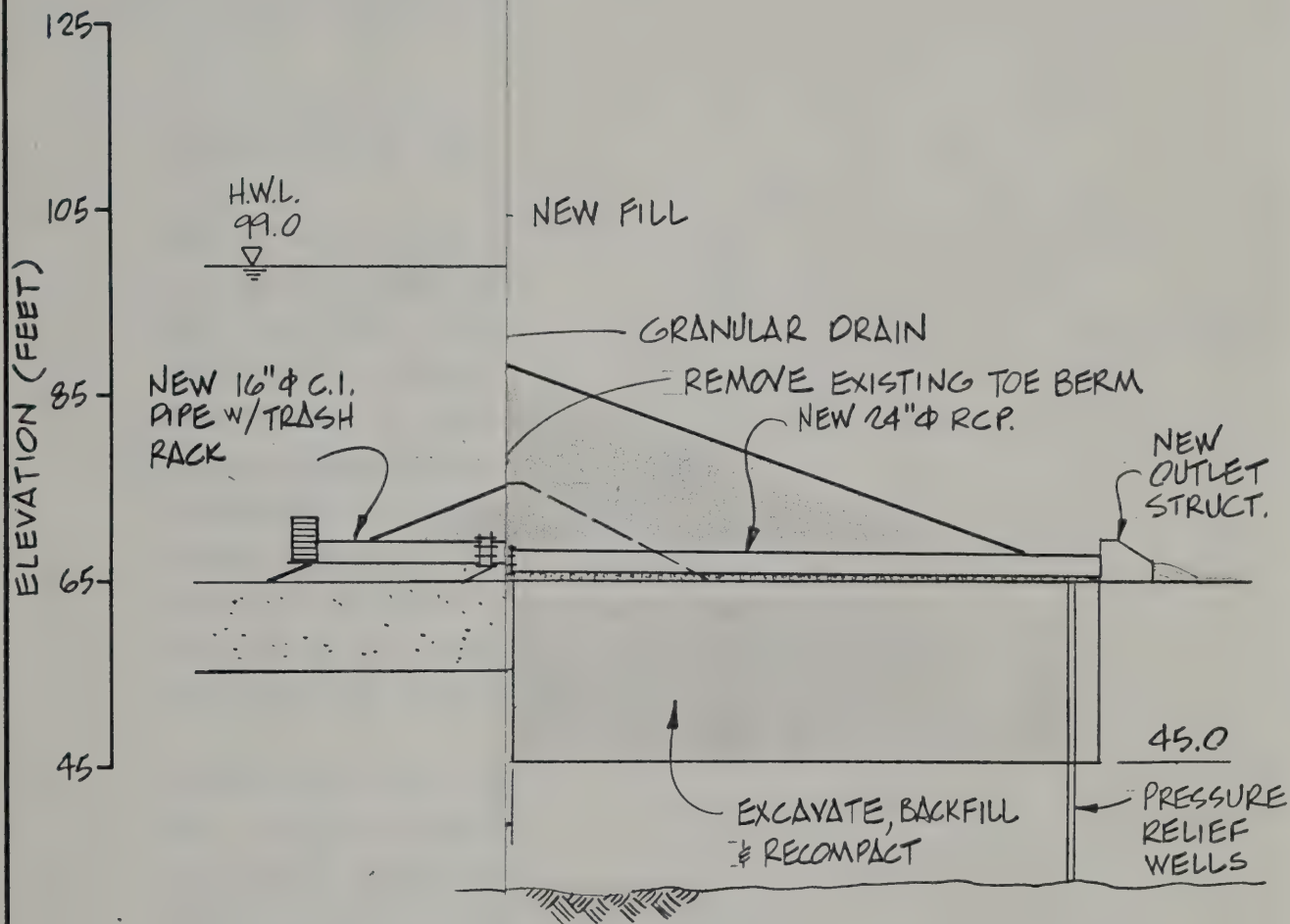
**PLAN VIEW
MAIN DAM REHABILITATION**

FIGURE 18



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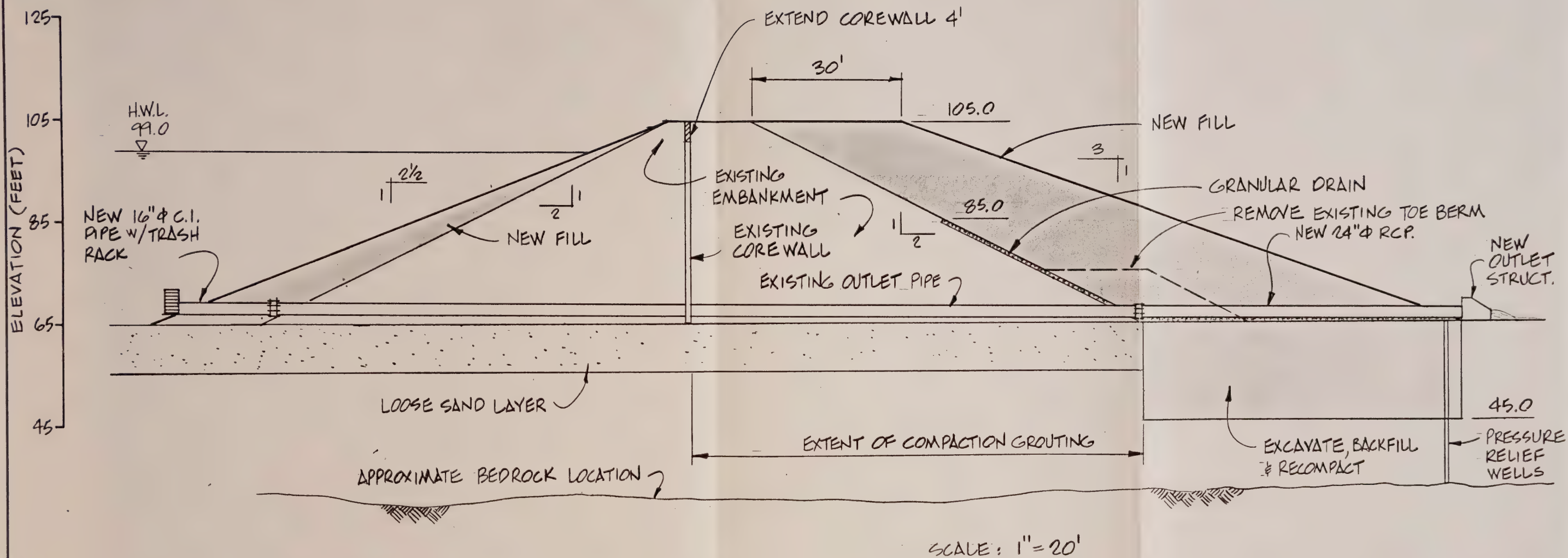
CHESSMAN DAM
REHABILITATION INVESTIGATION

CROSS SECTION
MAIN DAM REHABILITATION

FIGURE 19



MORRISON-MAIERLE, INC.
CONSULTING ENGINEERS



CHESSMAN DAM
REHABILITATION INVESTIGATION

CROSS SECTION
MAIN DAM REHABILITATION

FIGURE 19



4.4 COST ESTIMATES

Estimated cost for project rehabilitation includes all expected cost items regarding the Main Dam, Saddle Dam, Outlet Works, and Corewall. The preliminary cost estimate is based on the recommended rehabilitation plan in 1984 dollars which corresponds to a Construction Cost Index of 4118 as reported by the Engineering News Record. The costs are presented on Table 5.

Explanations for each major item are included below.

Compaction grouting - This item represents the single largest cost item in the rehabilitation of the Chessman project. It is also subject to a wide range of cost estimates due to the limited number of grouting contractors (nationwide) and the difficulty in determining quantities. In order to provide the best possible cost estimates Northern Engineering and Testing contacted the Hayward-Baker Company in Odenton, Maryland during March of 1984. This firm is one of the largest grouting contractors in the country and they were provided with plan and profile sheets and a description of the project. They provided an estimated mobilization, drilling and compaction grouting cost which was used for this estimate.

Subexcavation and compaction - It is proposed that the downstream toe area of the existing embankment be subexcavated and compacted rather than using expensive compaction grouting to densify the loose sand area. This will require the use of loader and dump truck equipment and the stockpiling of material. A cost of \$6.00 per cubic yard was used based upon Montana Department of Highways bid tabulations and the Dodge Guide to Public Works and Heavy Construction.

Other excavation and/or compaction - All other excavation and stockpiling or compaction earthwork was estimated at \$4.00 per cubic yard. This work would involve the use of a loader and dump truck to move the material to or from a stockpile area and compaction and watering equipment.

Table 5
PRELIMINARY COST ESTIMATE

CHESSMAN DAM REHABILITATION INVESTIGATION

<u>ITEM</u>	<u>DESCRIPTION</u>	<u>QUANTITY</u>	<u>UNIT</u>	<u>UNIT PRICE</u>	<u>AMOUNT</u>
MAIN DAM					
1.	Compaction grouting	1	EA	\$198,000.00	\$198,000.00
2.	Subexcavate, compact foundation	6,220	CY	6.00	37,320.00
3.	Place and compact downstream face	19,820	CY	4.00	79,280.00
4.	Clear and scarify slope	1	EA	3,200.00	3,200.00
5.	Drain material	1,690	CY	9.00	15,210.00
6.	Filter fabric	5,070	SY	2.00	10,410.00
7.	Remove and stockpile riprap	2,050	CY	6.00	12,300.00
8.	Place and compact upstream fill	4,960	CY	4.00	19,840.00
9.	Furnish and place riprap with blanket	3,070	CY	20.00	61,400.00
10.	6 inch diameter vertical wells	160	LF	15.00	2,400.00
11.	Grade and reseed slopes	1	ACRE	1,000.00	1,000.00
EXTEND OUTLET WORKS					
12.	Furnish and install 16" cast iron pipe	25	LF	40.00	1,000.00
13.	Furnish and install 24" RCP	70	LF	40.00	2,800.00
14.	New impact basin	5	CY	500.00	2,500.00
15.	Furnish and install trashrack	1	EA	2,000.00	2,000.00
EXTENSION OF COREWALL					
16.	Excavate and stockpile embankment	960	CY	4.00	3,840.00
17.	Furnish and place concrete with ties	90	CY	400.00	36,000.00
18.	Place and compact embankment	960	CY	4.00	3,840.00
SADDLE DAM					
19.	Earthwork and site preparation	2,750	CY	4.00	11,000.00
20.	Corewall inspection	470	CY	6.00	2,820.00
21.	Remove corewall to 99	12	CY	100.00	1,200.00
22.	Remove spillway	42	CY	100.00	4,200.00
23.	Membrane lining	1,325	SY	2.00	2,650.00
24.	Filter blanket	985	CY	9.00	8,865.00
25.	Class 3 riprap	1,480	CY	25.00	37,000.00
26.	Class 2 riprap	315	CY	20.00	6,300.00
SUBTOTAL					566,105.00
MOBILIZATION AT PERCENT				5.00	28,305.00
CONTINGENCIES AT PERCENT				20.00	113,221.00
CONSTRUCTION COST					707,631.00
ENGINEERING AT PERCENT				8.00	56,610.00
GEOTECHNICAL SUBCONTRACTOR AT					60,000.00
TOTAL ESTIMATED PROJECT COST					\$824,241.00
					\$825,000.00

Drain material - Graded aggregate drain material was estimated at \$9.00 per cubic yard based on MDOH bid tabulations. This material could eliminate the need for filter fabric, however, a filter fabric is recommended.

Filter fabric - A permeable filter fabric is recommended to control the piping of fines from the existing dam embankment into the proposed drain. A cost of \$2.00 per square yard was used based upon manufacturers price quotations.

Rock riprap - Slope protection for the upstream face of the Main Dam were estimated to cost \$25.00 per cubic yard based upon the class of riprap and bid tabs from the MDOH and the Rimini road repairs following the 1981 flood.

Cast iron pipe - To match the existing inlet pipeline, equally sized 16" diameter cast iron shall be added with a new trashrack assembly. A unit cost of \$40.00 per linear foot was based on the Dodge Guide.

Reinforced concrete pipe - Class V reinforced concrete pipe is proposed for the outlet conduit. A unit price of \$40.00 per linear foot was used in the cost estimate and was based on MDOH bid tabs.

Impact basin - An impact basin at the end of the outlet conduit was based upon a standard stilling basin design used by the Soil Conservation Service. This structure will reduce the outflow velocities to a non-erosive value. A unit price of \$500.00 per cubic yard was used to estimate the cost of forming, resteel placement and concrete in the structure.

Earthwork at Saddle Dam - Site preparation and earthwork at the Saddle Dam would include clearing the upstream and downstream slope areas of trees and vegetation and the excavation to proper subgrade for the placement of the filter fabric and riprap slope protection.

Corewall inspection at Saddle Dam - Three observation pits are proposed at Saddle Dam to inspect the condition of the corewall and verify that it extends to bedrock. These would be accomplished following the site work to limit the amount of excavation required and would use a backhoe. A unit price of \$6.00 per cubic yard was used to estimate the backhoe work.

Corewall removal - The existing corewall is approximately 105 feet long with a crest at elevation 102 feet and would be cutoff to elevation 99 feet. A unit price of \$100.00 per cubic yard was used to estimate the cost of cutting and removing the unreinforced concrete wall.

Filter fabric - A permeable filter fabric is recommended to control the piping of fines from the Saddle Dam subgrade to the proposed riprap face protection. A cost of \$2.00 per square yard was used based upon manufacturers price quotations.

Riprap face protection - Protection of the downstream face would use Class III riprap and Class II would protect the upstream face. Class III slope protection would range in gradation from 0.77 to 2.82 feet, while Class II would range from 0.61 to 2.00 feet. Protection of the upstream face is required to prevent erosion which would occur from the velocity of approach. Cost of providing and placing riprap protection at the Saddle Dam for Class III and II was estimated at respectively; \$25.00 and \$20.00 per cubic yard.

Mobilization - A 5 percent cost was used to cover the expense of moving and setting up the contractor and subcontractors at the Chessman project site. This costs was based upon Bureau of Reclamation bid tabs for dam construction.

Contingencies - A 20 percent contingency was used to estimate the uncertainties in the quantities and unit prices estimated and small items not covered in the cost estimate. The largest uncertainty is the compaction grouting. Both the quantities and estimated cost are

subject to some unknowns. The actual extent of the unconsolidated sand lense in the foundation will not be known until the grouting program is completed.

Engineering - The fee for providing final engineering and construction inspection was estimated at 12 percent. The estimated engineering fee reflects the involved nature of a rehabilitation project.

Geotechnical investigation - Geotechnical engineering services for the project are substantial due to the nature of the problem in the foundation. A \$20,000 fee was estimated by Northern Engineering and Testing to cover the cost of providing the following geotechnical services for final design; (1) define extent of loose foundation zones including field drilling, laboratory testing and analysis, (2) investigate and locate suitable borrow areas for embankment construction, and (3) provide report of data and recommendations for final design and specifications.

A \$40,000 fee was estimated to cover the cost of providing geotechnical services during construction for the following items; (1) observation of grouting and foundation compaction (would require full-time geotechnical engineer on-site), (2) verification of densification by grouting which will require field drilling and standard penetration tests, (3) observation and density testing of embankment construction, and (4) review of data and analysis during construction to ensure that rehabilitation of project is meeting design including reports on findings.

Note that the cost of providing construction inspection is heavily dependent upon the actual construction schedule.

4.5 SCHEDULING

An estimated construction schedule is shown on Figure 20 which would require a period from the first of June to the end of October. Final engineering and design would require a minimum of six months prior to construction for design, preparation of bid documents and bid letting. Note that the six month period must include a period during which the geotechnical field investigation for final design can be accomplished.

On Figure 20 the actual construction time is represented by the heavy portion of the line for each task. The thin line represents the slack time for each task or the maximum allowable period that each task may be accomplished. The critical path through the project shown on Figure 20 is the site preparation grouting and installation of the drains and new downstream face. This path is estimated to take 90 construction days or the period from June 1st to October 7th. Note that this schedule assumes normal spring and summer conditions and a construction contractor with adequate equipment and manpower to accomplish the work expeditiously. A brief explanation of each major item on the Schedule is included below.

Start - A starting date for construction on June 1st is proposed.

Ewspill - Earthwork for site preparation at Saddle Dam spillway.

Exc - Excavate and stockpile embankment surrounding corewall.

Foundcom - Subexcavate and compact foundation at downstream toe of Main Dam.

Extendcw - Form and place 5 foot corewall extension.

Cutoff - Remove Saddle Dam corewall to elevation 99 feet.

Blanket - Place filter blanket and graded filter material at Saddle Dam.

QKPLACE - Foundation area under outlet compacted successfully.

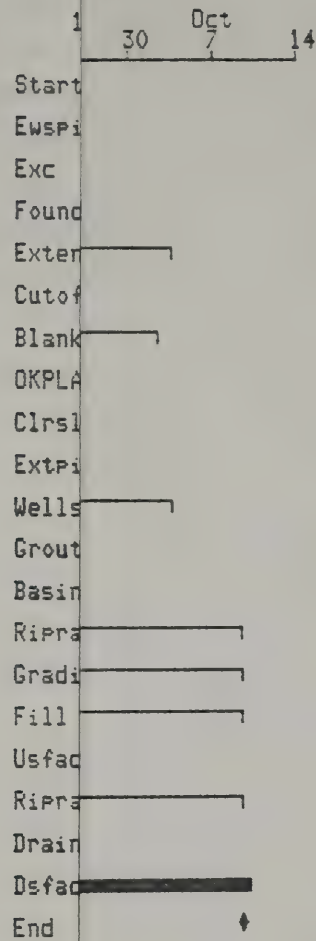
Clrslope - Clear and scarify slopes at Main Dam.

Extpipe - Extend cast iron and concrete pipes upstream and downstream.

Wells - Install 6" diameter vertical wells at toe of dam embankment.

Grout - Compaction grouting of loose sand layer in foundation.

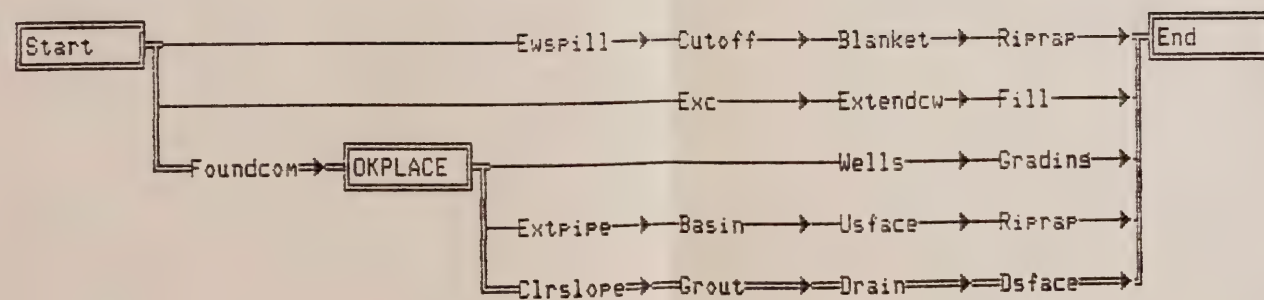
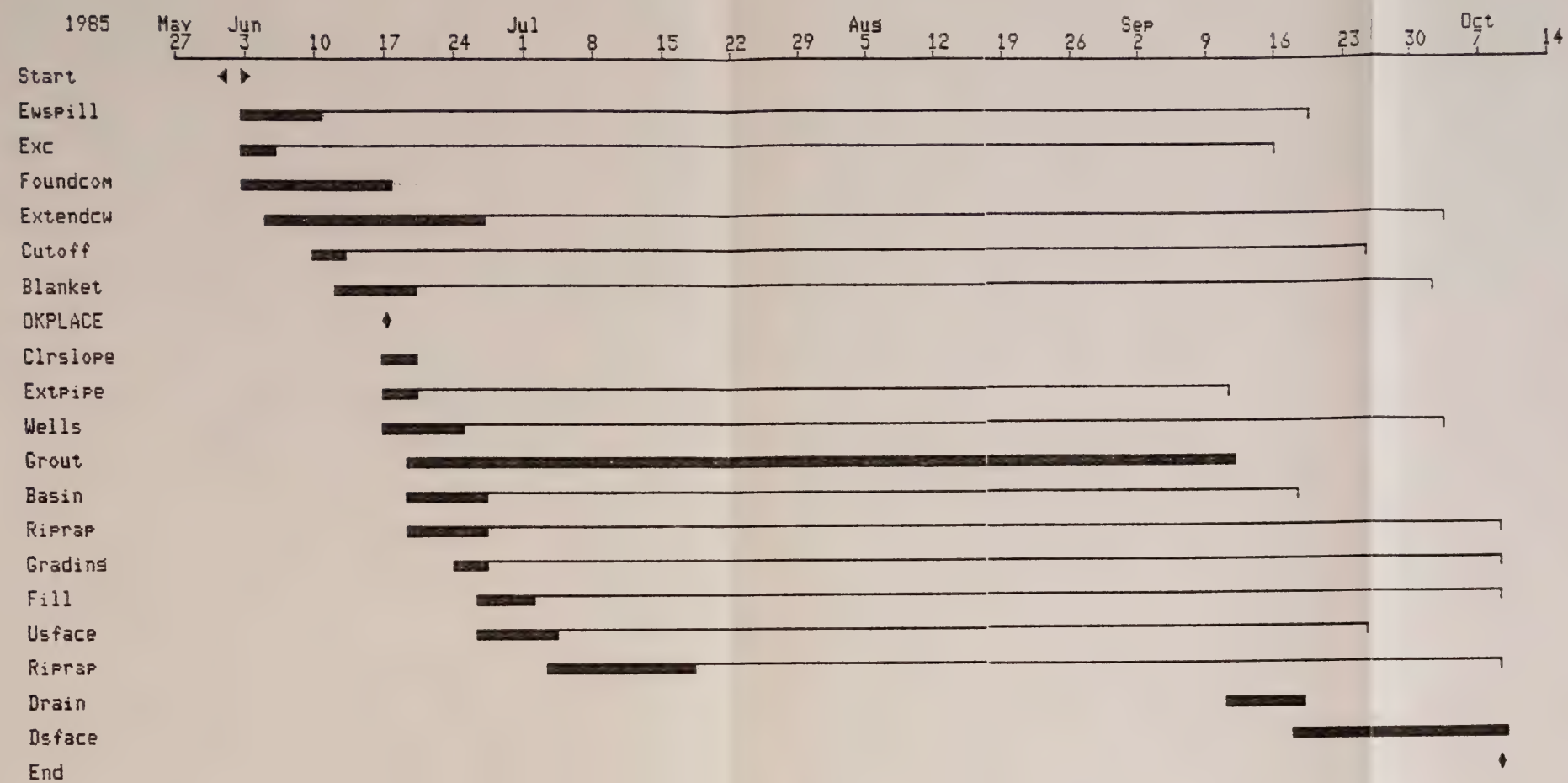
Basin - Install a new impact basin at downstream end of concrete outlet pipe.



CHESSMAN DAM
 REHABILITATION INVESTIGATION
CONSTRUCTION SCHEDULE
 AND
PROJECT ACTIVITY DIAGRAM
 FIGURE 20



MORRISON-MAIERLE, INC.
 CONSULTING ENGINEERS



CHESSMAN DAM
 REHABILITATION INVESTIGATION
 CONSTRUCTION SCHEDULE
 AND
 PROJECT ACTIVITY DIAGRAM
 FIGURE 20





Riprap - Place filter blanket and riprap at Saddle Dam.

Grading - Finish grading and seeding of all exposed slopes.

Fill - Backfill and compact trench surrounding corewall.

Usface - Remove riprap, stockpile and flatten upstream slope.

Riprap - Place filter blanket and riprap at Main Dam.

Drain - Install drain on downstream face of Main Dam.

Dsface - Place and compact new embankment on downstream face of Main Dam.

End - Estimated completion date of October 7.

APPENDIX

CHES4/E



Northern

Engineering
and Testing, Inc.

528 Smelter Ave.
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December 1, 1983

Morrison - Maierle, Inc.
Consulting Engineers
PO Box 6147
910 Helena Avenue
Helena, MT 59604

ATTENTION: Mr. Harold Eagle, P.E.

SUBJECT: Geotechnical Investigation
Chessman Dam

Gentlemen:

At your request and in accordance with our agreement, we have completed geotechnical investigations at the Chessman Dam rehabilitation project, near Helena, Montana. The purpose of our investigation was to evaluate the stability of the embankments and to provide recommendations to rehabilitate the embankment and spillway to meet current safety standards. We have met with you and the City of Helena Steering Committee on several occasions to review our findings as the investigation progressed. The enclosed report summarizes our findings and presents alternates for remedial action.

Chessman Reservoir is formed by the main dam, which is about 40 feet high, and a saddle dam, about 15 feet high. Both dams have a concrete cutoff extending through earth embankment. The embankments are located in an active seismic area.

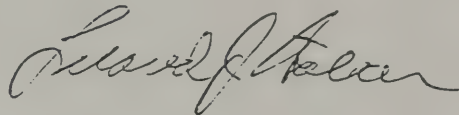
Both the main dam and the saddle dam are composed of sand or clay fill. The main dam is generally underlain by clay, sand or gravel, and then igneous rock. There is a continuous layer of loose sand near the base of the embankment, at the south abutment of the dam. The concrete cutoff at the main dam does not extend to bedrock, and there is considerable seepage beneath the dam. At the saddle dam, it appears the embankment and cutoff are founded in bedrock.

Analysis indicates the loose sand layer beneath the south portion of the dam will be subject to liquefaction under earthquake loading. Our analysis has been reviewed by a seismic consultant, Dr. H. Bolton Seed, an internationally recognized expert in earthquake engineering. Additionally, the slope stability of the embankment is marginal, and does not meet current safety standards.

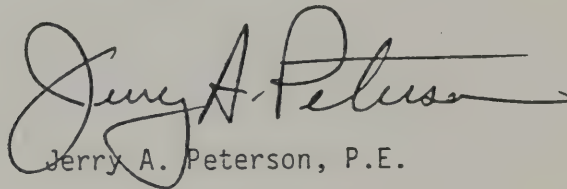
We have developed alternate procedures to densify the loose sand layer below the dam and improve the slope stability of the embankment. These include the use of compaction grouting to densify the sand layer, and providing a toe berm, flattening the upstream slope, and installing drains and pressure relief wells to improve stability.

We have enjoyed working with you on this interesting project, and look forward to continued involvement as plans for rehabilitation are developed. If you have any questions, or we can be of further assistance, please contact us at your convenience.

Respectfully submitted,



Leland J. Walker, P.E.



Jerry A. Peterson, P.E.

Enclosure
In quadruplicate
LJW/JAP/s

RECOMMENDATIONS

General

1. All fill and backfill should be approved by our soils engineer, placed in uniform lifts, and compacted to at least 95 percent of the maximum dry density determined by ASTM D698.
2. Prior to rehabilitation, borrow source investigations should be conducted to locate suitable materials for embankment and riprap. After an embankment borrow source has been located, strength tests should be conducted to confirm the strength parameters used in our analysis. We are available to outline and conduct a borrow source investigation.

Main Dam Embankment

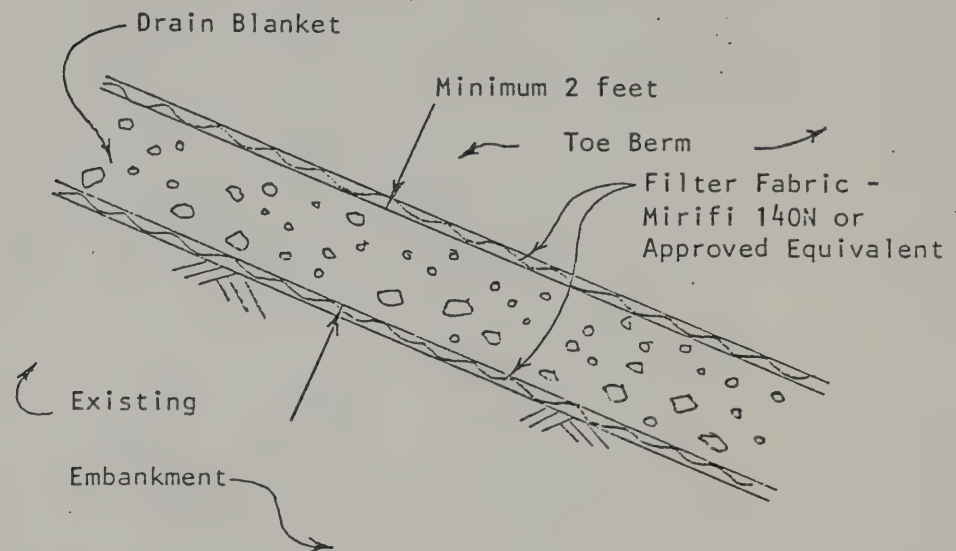
Alternate 1

3. Densify all of the loose sand layer and construct a toe berm as discussed in the Engineering Analysis.
 - A. The toe berm should be 20 feet wide at the top, have a top elevation of 85 feet and a slope of 3 horizontal to 1 vertical.

B. Construct a filter fabric and rock drain, or a graded aggregate drain between the existing embankment and the toe berm.

a. The drain should be a minimum of 2 feet thick perpendicular to the slope and should daylight at the toe of the dam.

b. If a filter fabric and rock drain is used, a fabric similar to Mirifi 140N, or approved equivalent, should be used. The rock used in the drain should have a maximum size of 3 inches, with no more than 5 percent minus No. 200 sieve size. The configuration of the drain should be as shown below:

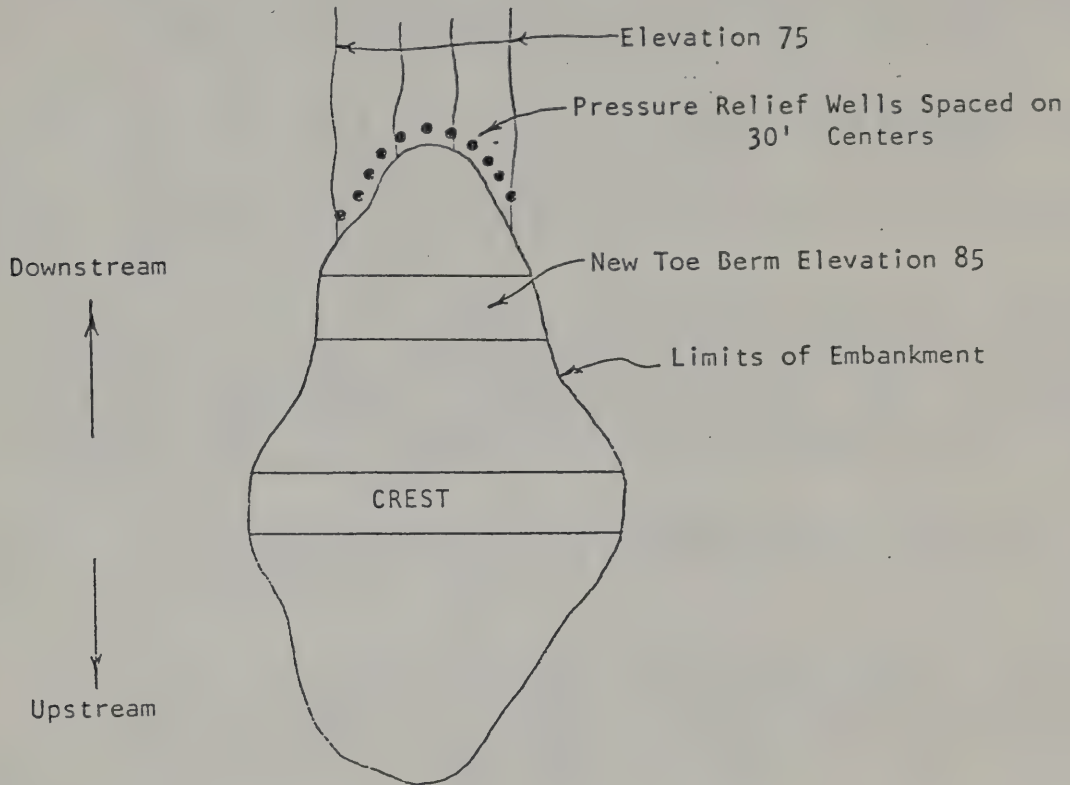


c. If a graded aggregate drain is used in lieu of the filter fabric and rock drain, it will be necessary to establish the grading requirements, after a borrow source has been selected. A processed aggregate will probably be required.

d. The top of the drain should extend up to elevation 83.

C. Flatten the upstream slopes to 2-1/2 horizontal to 1 vertical.

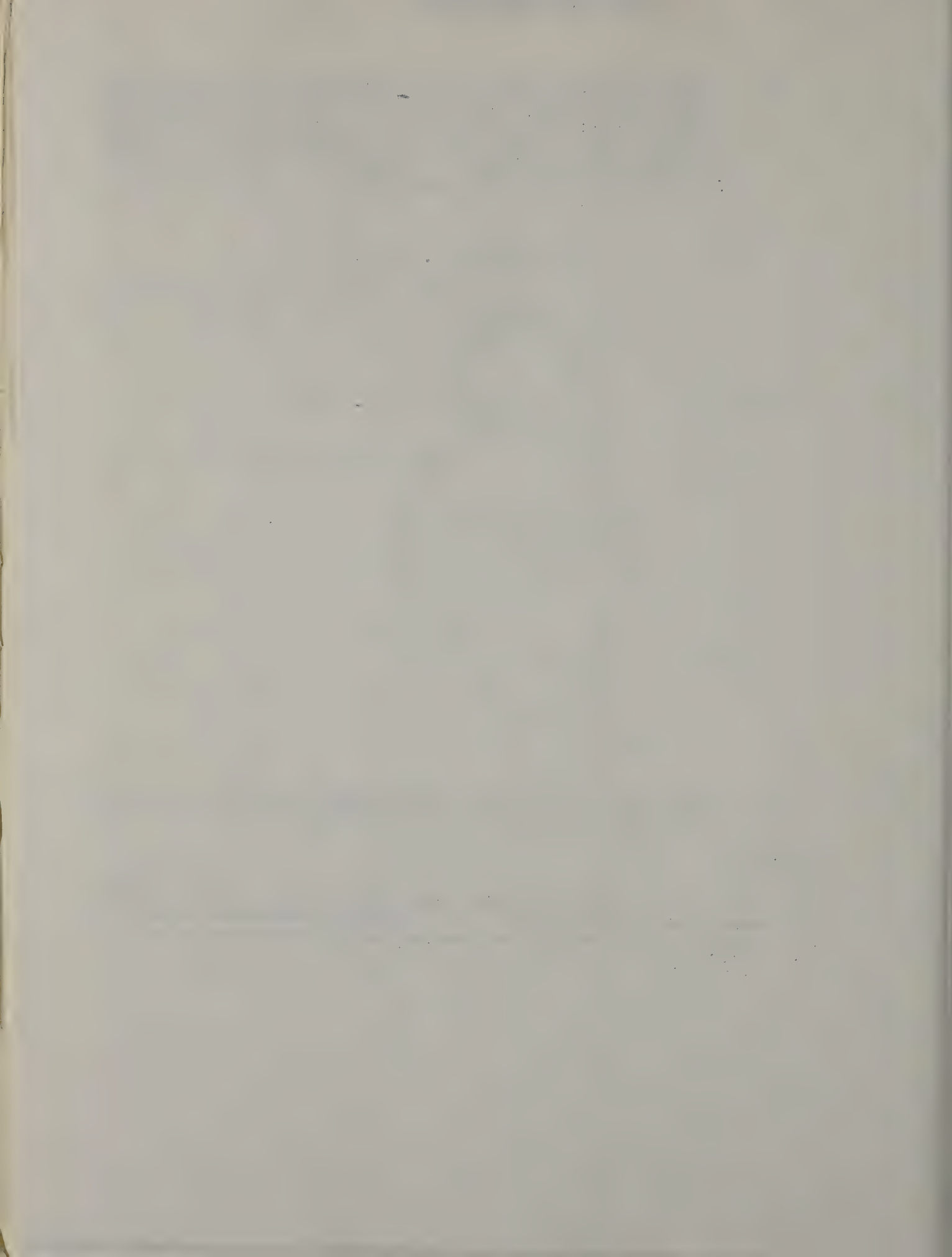
- D. Provide pressure relief wells around the toe of the embankment. Wells should be a minimum of 4 inches in diameter, at least 20 feet deep, and spaced on 30 foot centers. The wells should extend around the toe to elevation 75 on either abutment, and be allowed to drain freely. The configuration of the wells should be as shown below:



- E. Additional test borings should be made to define the upstream and lateral extent of the loose sand deposit.

Alternate 2

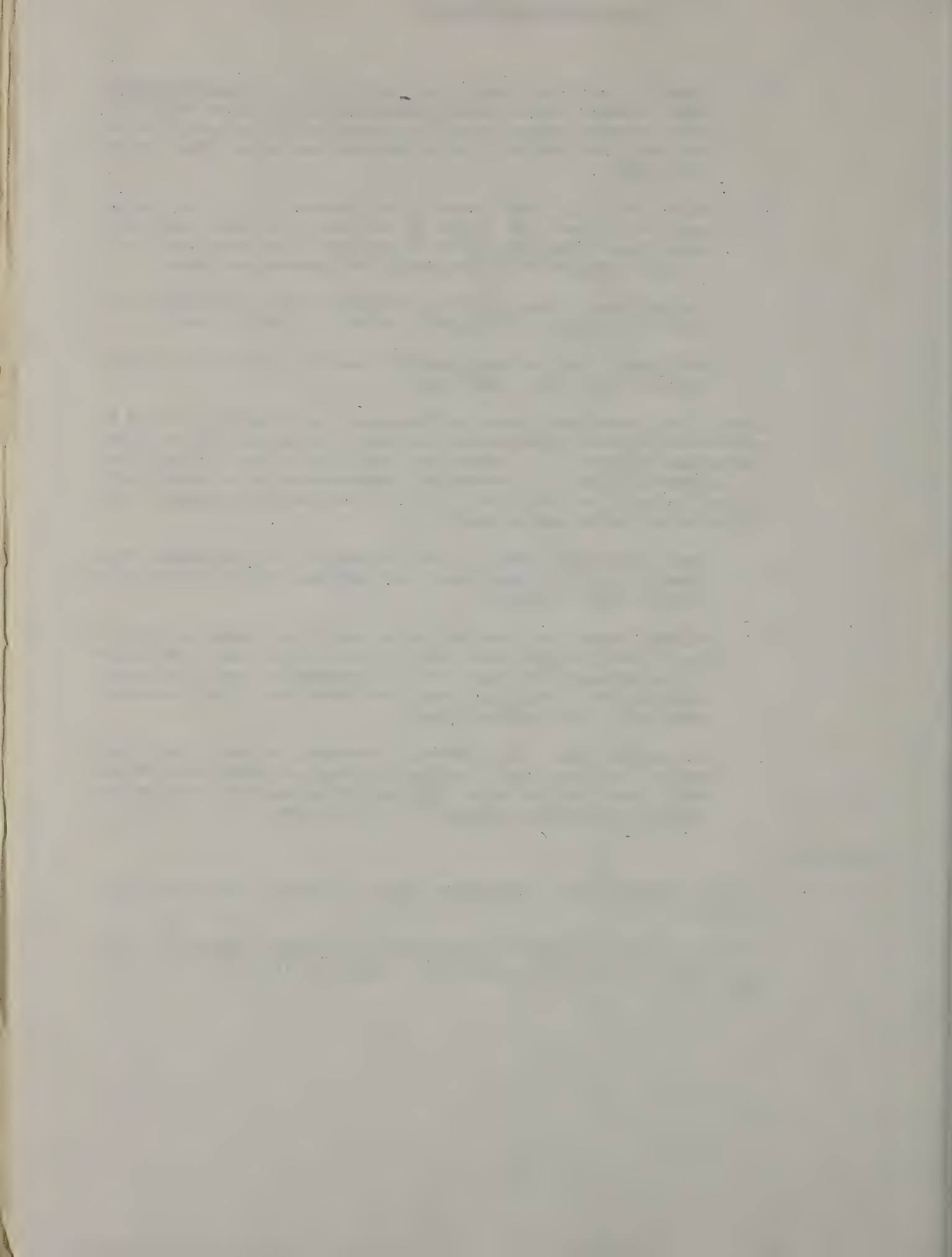
4. Densify the loose sand layer downstream of the concrete cutoff, widen the crest by 30 feet, and flatten the downstream slope to 3 horizontal to 1 vertical as discussed in the Engineering Analysis.



- A. Construct a filter fabric and rock drain, or graded aggregate drain between the existing embankment and the new fill. The drain should extend upward to elevation 85 and daylight at the toe. For design, the recommendations under item 3B are applicable.
 - B. Where the loose sand layer is not present, it is not necessary to extend the crest by 30 feet. In this area, the downstream slope should be flattened to 3 horizontal to 1 vertical and the drain constructed as recommended above.
 - C. The upstream slopes should be flattened, and pressure relief wells installed, as outlined in items 3, C and D above.
 - D. Additional test borings should be made to define the lateral extent of the loose sand layer.
5. Densify the loose sand layer as discussed for Alternates 1 or 2 by overexcavating and replacing the sand with compacted fill or using compaction grouting. If compaction grouting is used, a qualified and competent specialty contractor, experienced with these techniques, should be selected. For cost estimating purposes, the following estimates may be used:
- A. The approximate area to be grouted may be estimated from Drawing 83-575-1. Additional test borings will be required to better define the limits.
 - B. Assume compaction grouting holes would be spaced on about 7 to 10 foot centers over the portion of the loose sand layer to be grouted. For each hole, it appears it would be necessary to drill to the loose sand layer and inject approximately one cubic yard of grout.
 - C. For preliminary cost estimating purposes, assume it will cost about \$10.00 per foot for the drilling, and about \$200.00 per cubic yard for the grout in place. More accurate costs may be obtained by contacting specialty contractors.

Saddle Dam

- 6. Flatten upstream and downstream slopes to 2-1/2 horizontal to 1 vertical.
- 7. After a spillway design and configuration has been selected, we are available to provide any additional geotechnical design parameters required.

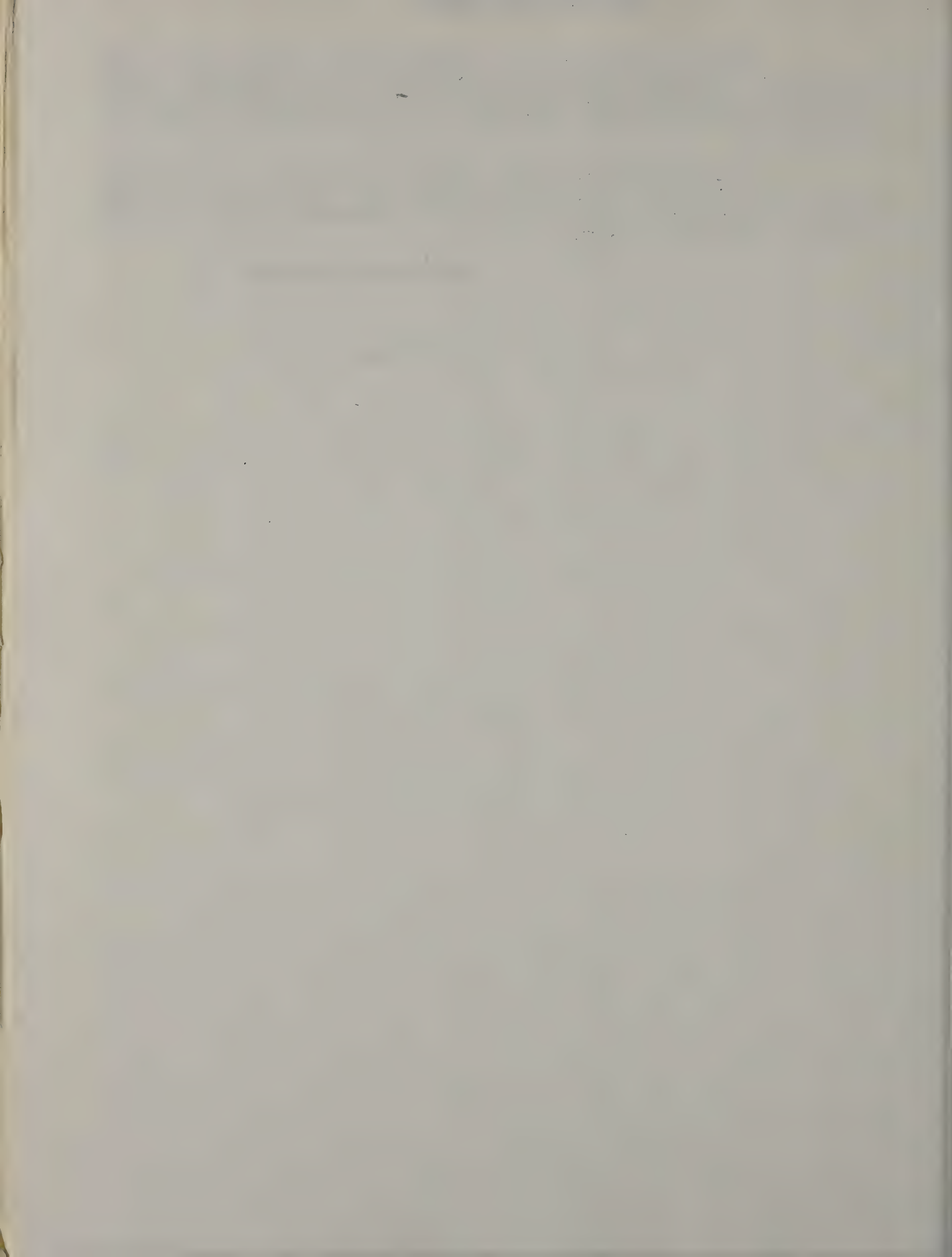


The conclusions reached and recommendations provided in this report are based on information from the test borings, and the assumptions stated. Additional investigations will be required. If these investigations, or construction indicates different conditions, it may be necessary to modify our recommendations.

If changes in the nature, design, or location of the dam are planned, the recommendations contained in this report shall not be considered valid unless the changes are reviewed and the recommendations of this report modified or verified in writing.

Respectfully submitted,

Al Stilley, P.E.



R E F E R E N C E S

1. "Phase I Inspection Report, National Dam Safety Program, Beaver Creek and Buffalo Creek Basins, Chessman Main Dam and Chessman Saddle Dam Near Rimini, Montana, MT-1090 and MT-1562," Morrison - Maierle, Inc., Consulting Engineers, Helena, MT, June, 1980.
2. Qamar, Anthony I., and Stickney, Michael C. "Montana Earthquakes, 1869 to 1979, Historical Seismicity and Earthquake Hazard," Memoir 51, Montana Bureau of Mines and Geology and the University of Montana, Butte, Montana, 1983.
3. Stickney, Michael and Bingler, E.C., "Earthquake Hazard Evaluation of the Helena Valley Area, Montana," Open File Report 83. Montana Bureau of Mines and Geology, Butte, Montana, August 1981.
4. Seed, H.B., Idriss, I.M., Arango, I., "Evaluation of Liquefaction Potential Using Field Performance Data," Journal of Geotechnical Engineering, American Society of Civil Engineers, Vol. 109, No. 3, March, 1983, pp. 458 - 482.
5. "Recommended Guidelines for Safety Inspection of Dams," Department of the Army, Corps of Engineers, Washington, D.C., 1972.



Northern

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(406) 248-9161

June 15, 1984

Morrison-Maierle, Inc.
P O Box 6147
Helena, MT 59604

SUBJECT: Geotechnical Design Data
Chessman Dam Project.

Gentlemen:

In response to questions during our meeting with the Chessman Dam Steering Committee in Helena on May 23, 1984, we are providing additional reference information. Enclosed is the following information:

1. Plate 21 - Grain size distribution curves for samples not provided in our report of December 1, 1983.
2. Table 1 - Summary of field and laboratory tests on sand samples.
3. List of References - (Numbers in parentheses in following text refer to numbered reference).
4. Corps of Engineers - Recommended Guidelines for Safety Inspection of Dams.
5. Compaction Grouting References
6. Summary Site Seismicity Evaluation, Mystic Lake Dam Project.

The questions asked in the meeting of May 23, 1984 are listed and addressed below.

1. Liquefaction Potential - We were requested to review the field information and provide minimum ground accelerations at which liquefaction could be expected.

Reference (1) was used to determine susceptibility to liquefaction. With this procedure, a cyclic stress ratio for a particular sand

Morrison-Maierle, Inc.
Helena, MT

June 15, 1984
Page 2

layer is calculated as:

$$\frac{\gamma_u}{\sigma'_0} = 0.65 \frac{a_{\max}}{g} \times \frac{\sigma_0}{\sigma'_0} \times r_d, \quad \text{where}$$

$a_{\max}$ = maximum acceleration at ground surface, percentage of gravity.

σ_0 = total overburden pressure on sand layer.

σ'_0 = initial effective pressure on sand layer.

r_d = stress reduction factor ranging from 1 at ground surface to 0.9 at a depth of 30 feet.

g = acceleration of gravity.

With this procedure, a cyclic stress ratio causing liquefaction is determined for a particular sand layer, based on the N value, Figure 3(a) (1). If this value is set equal to the equation shown above, the minimum acceleration causing liquefaction can be determined. Potentially liquefiable zones in the sand were identified in our original report and are included on the attached Table 1. For instance, at Drill Hole 20, 25.5' to 27.0', $N = 5$, depth to water = 11'.

$$\sigma_0 = 25 \times 120 = 3000 \text{ psf}$$

$$\sigma'_0 = 11 \times 120 + (25 - 11) \times (120 - 62.4) = 2130 \text{ psf} = 2.13 \text{ ksf}$$

$$C_n = 1.0 \text{ from Figure 4}$$

$$N_1 = C_n \times N = 5$$

$$\frac{\gamma}{\sigma'_0} = 0.05 \text{ from Figure 3(a) with } N_1 = 5$$

$$r_d \text{ (extrapolated)} = 0.92$$

Substituting in above

$$0.65 \frac{a_{\max}}{g} \times \frac{3000}{2130} \times 0.92 = 0.05 \text{ or}$$

$$a_{\max} = 0.06 g.$$

Evaluation of the remaining data for the loose sand layer generally indicates accelerations in the range of 0.05 g to 0.07 g would be required to cause liquefaction. It should be noted that generally the sands in the loose layer do not classify as silty sands, and the N value correction listed in the reference cannot be applied. In order to classify as a silty sand for liquefaction evaluation, it is necessary that $D_{50} < 0.25 \text{ mm}$. The D_{50} values are tabulated in Table 1 and may be obtained from the grain size distribution plates in our original report.

Morrison-Maierle, Inc.
Helena, MT

June 15, 1984
Page 3

References (2) indicate a maximum horizontal acceleration of 0.118 g was recorded in Helena during the Magnitude 6, October 31, 1935 earthquake. Reference (2) also indicates a Magnitude 6 earthquake in the Helena-Ovando region has a recurrence interval of 70 years, based on statistical analysis of past events. Reference (3) also indicates that simultaneous rupture along all fault segments of the Helena Valley fault zone would produce an earthquake in the magnitude 7.0 to 7.5 range. For these reasons, it was concluded there was a high possibility of liquefaction and additional analysis to determine the probability was not conducted.

A seismic coefficient of 0.15 was used in the pseudostatic earthquake analysis. This value was selected after consultation with Dr. Seed. It is slightly higher than the 0.1 value suggested by the Corps of Engineers (4) for a Zone 3 earthquake area. It should be noted that the seismic coefficient used in the pseudostatic analysis is not the same as the maximum ground acceleration which can be expected at a site. The 0.15 value also reflects our judgement after detailed studies at other nearby sites with similar geologic settings (5).

2. Strength Parameters used in Stability Analysis - We were requested to provide backup data for selection of strength parameters used in the stability analysis.

The Corps of Engineers, "Recommended Guidelines for Safety Inspection of Dams," a copy of which is attached, was used in selecting the strength parameters for stability analysis, and the minimum acceptable factors of safety for each reservoir loading condition. The reservoir conditions, minimum factors of safety, and strength parameters to use are summarized below:

Case	Loading Condition	Minimum Factor of Safety	Shear Strength Parameters to Use
I	Sudden drawdown from spillway crest	1.2	Minimum composite of drained and undrained strength parameters.
II	Partial pool with horizontal steady state seepage	1.5	Average of drained and undrained parameters for undrained less than drained - Use drained for undrained greater than drained.

Morrison-Maierle, Inc.
Helena, MT

June 15, 1984
Page 4

Case	Loading Condition	Minimum Factor of Safety	Shear Strength Parameters to Use
III	Steady state seepage from spillway crest	1.5	Same as Case II
IV	Earthquake - Cases II and III with seismic loading	1.0	Same as Case II or III

In evaluating the strength data, the consolidated, undrained triaxial tests with pore pressure measurement were considered as the undrained tests, and the direct shear tests were considered as the drained tests. These test results are summarized below:

Summary of Direct Shear - Drained Tests

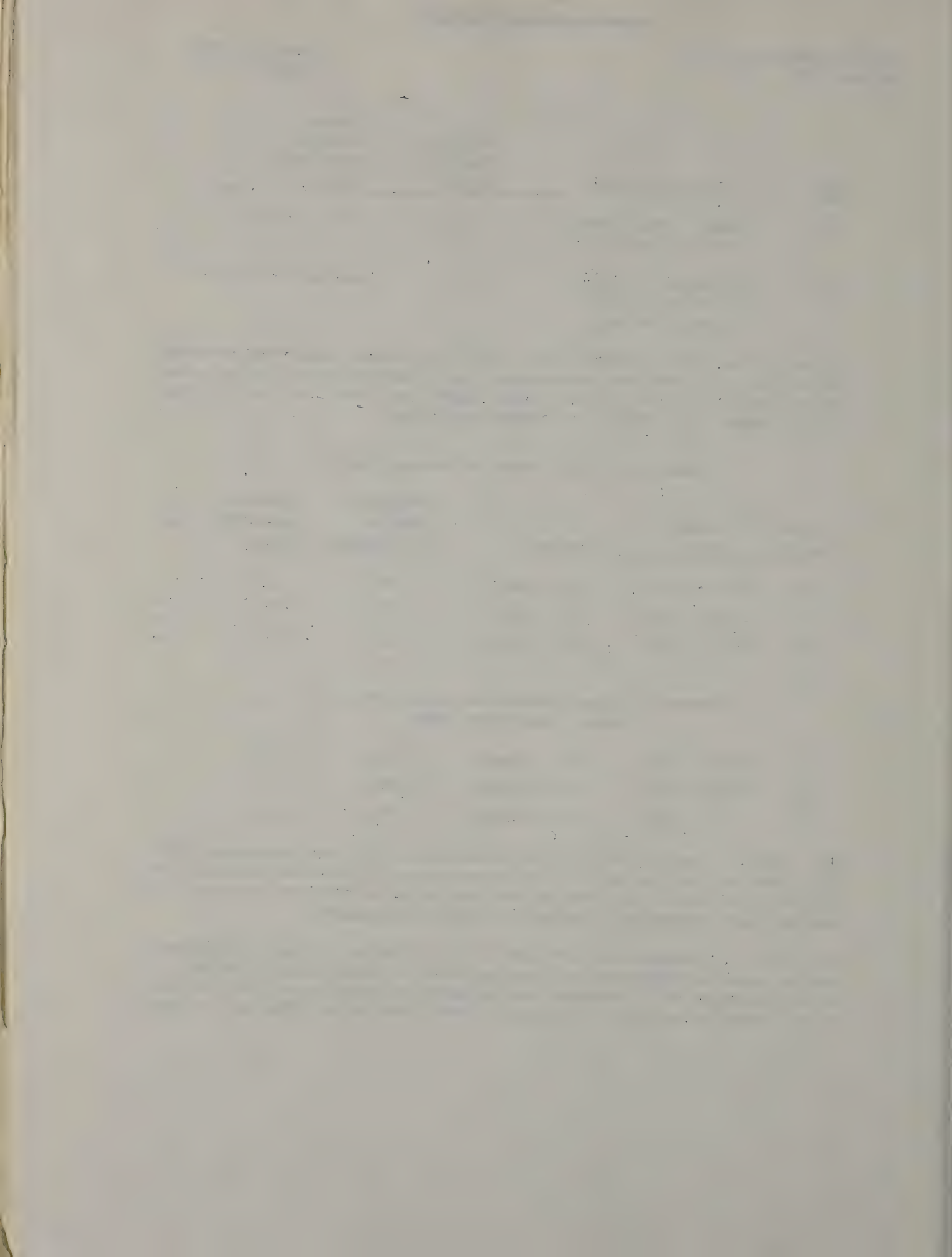
Drill Hole	Depth, feet	Material	Friction Angle ϕ , degrees	Cohesion Intercept $\bar{c}$, psf
6	14.3 - 14.5	FILL, Sand	40	0
9	9.2 - 9.8	FILL, Sand	30	1700
9	19.3 - 19.5	FILL, Sand	28	950

Summary of Consolidated Undrained Triaxial Tests - Undrained Tests

8	33.5 - 43.5	FILL, Sand	39	0
8	43.5 - 45.5	CLAY, Silty	26	0
13	18.0 - 20.0	CLAY, Silty	18	300

For Case I, evaluation of the above data for the embankment fill indicates a friction angle of 28 degrees and a cohesion intercept of 0 should be used. For the foundation soil, a friction angle of 18 degrees and a cohesion intercept of 300 psf was used.

For Cases II through IV, for the fill, a friction angle of 28 degrees and a cohesion intercept of zero was used. For the foundation soil, a friction angle of 26 degrees and a cohesion intercept of 300 psf was used. These values may be slightly lower than strict adherence to the



Morrison-Maierle, Inc.
Helena, MT

June 15, 1984
Page 5

guidelines would indicate but in our judgement, they are reasonable for the material types and there are several conditions existing which contribute to instability. These conditions include 1) high artesian pressures in the foundation and embankment, 2) layering in the embankment soils, and 3) the non-homogeneous nature of the foundation (See Table I for a summary of "N" values and particle size results).

3. Control of Compaction Grouting - It was requested that we address whether compaction grouting could be controlled to achieve densification of the loose layer.

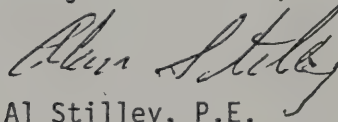
Compaction grouting is different from chemical grouting or the normal grouting performed to limit water flow through a soil or rock. A near zero slump soil-cement mixture is used in compaction grouting. The grout remains as a distinct homogeneous mass during injection. Densification is achieved through the process of displacement. A verification program would be incorporated into the grouting program to determine that the loose layer had actually been densified. Compaction grouting has successfully been used for about the last 20 years and methods have been developed for the design and control of the grouting program. Depending upon the alternate selected, additional test borings would be performed prior to finalizing the grout program, to better define the limits of the loose sand layer. We have included references, better explaining the process of compaction grouting, as well as information on other projects.

If you have questions, or if we can be of further assistance, please contact us at your convenience.

Respectfully submitted,



Jerry A. Peterson, P.E.

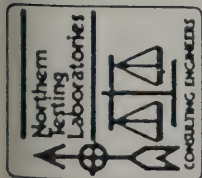


Al Stilley, P.E.

AS:rmr -

Enclosures in triplicate

cc: Mr. Doug McClelland
U.S. Forest Service
Missoula, MT



Project: Chessman Dam
Location: Drill Hole No. 20
Classification: FILL; SAND, Silty (Unified System)
Moisture Content _____ %
Liquid Limit _____ %

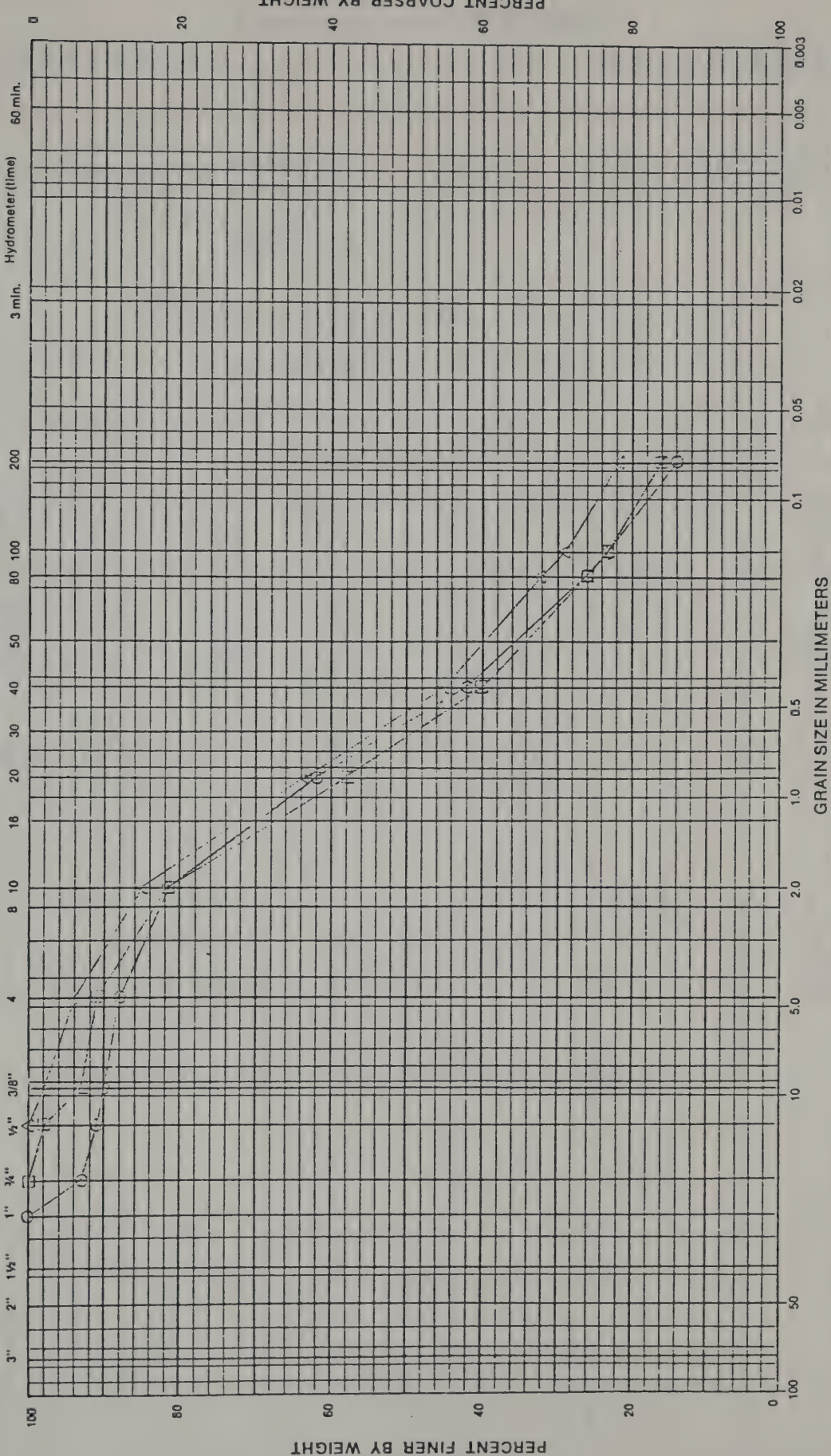
Depth _____
O 4.0 - 5.5
Δ 12.5 - 14.0
□ 17.5 - 19.0

Job No. 83-575

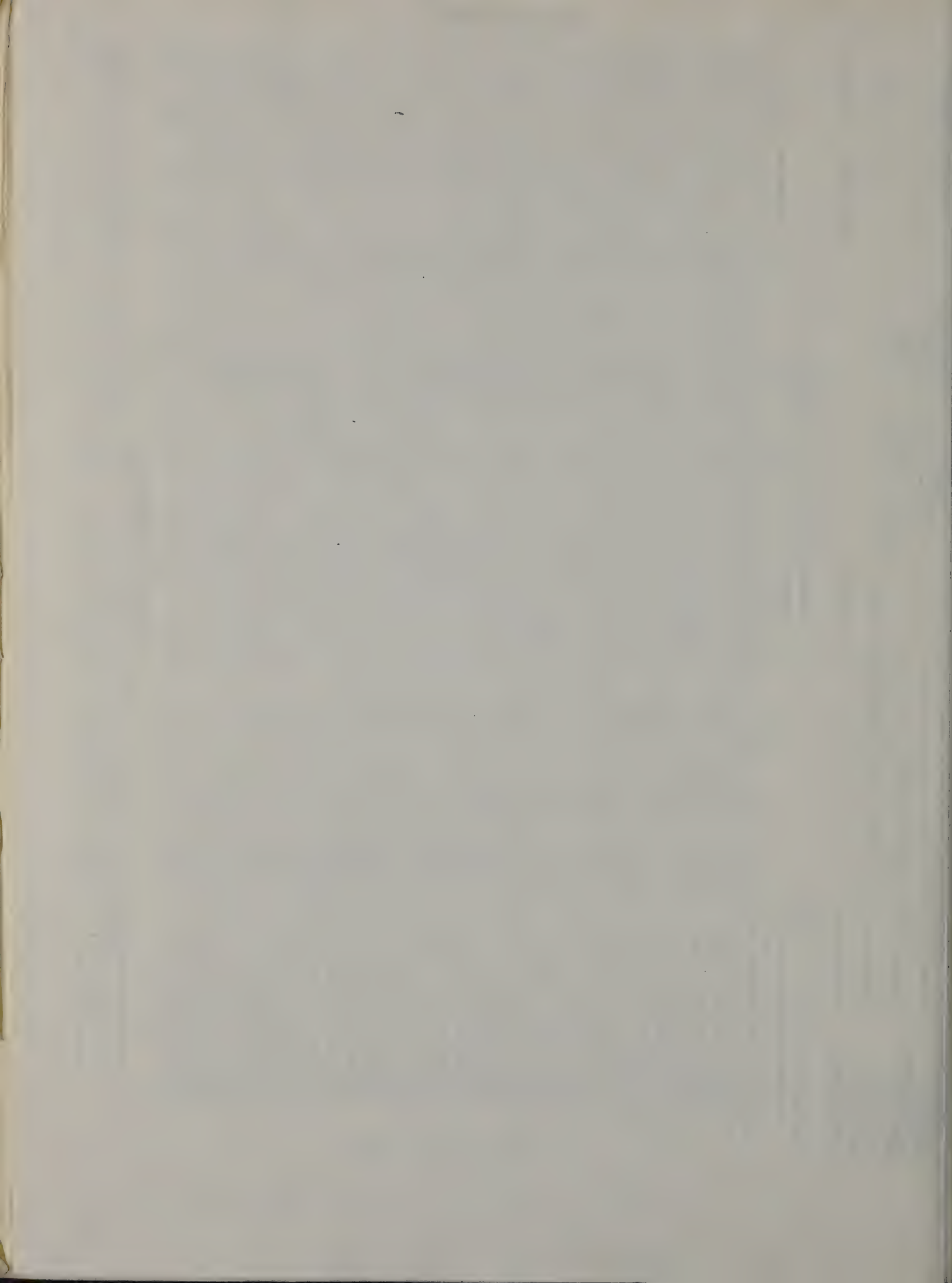
Plasticity Index _____ %

Coefficient of Uniformity = $C_U = \frac{D_{60}}{D_{10}} = \frac{100}{100} = 1.0$

Coefficient of Curvature = $C_Z = \frac{(D_{30})^2}{D_{10} \times D_{60}} = \frac{100^2}{100 \times 100} = 1.0$



Gravels		Sand		Fines	
Coarse	Fine	Coarse	Medium	Fine	Clay



STANDARD COMPUTATION SHEET

PROJECT Chessman JOB NO. 83-575
 PURPOSE _____ SHEET _____ OF _____
 COMPUTED BY JP CHECKED BY _____ DATE 6/4/84

CHESMAN DAM
83-575

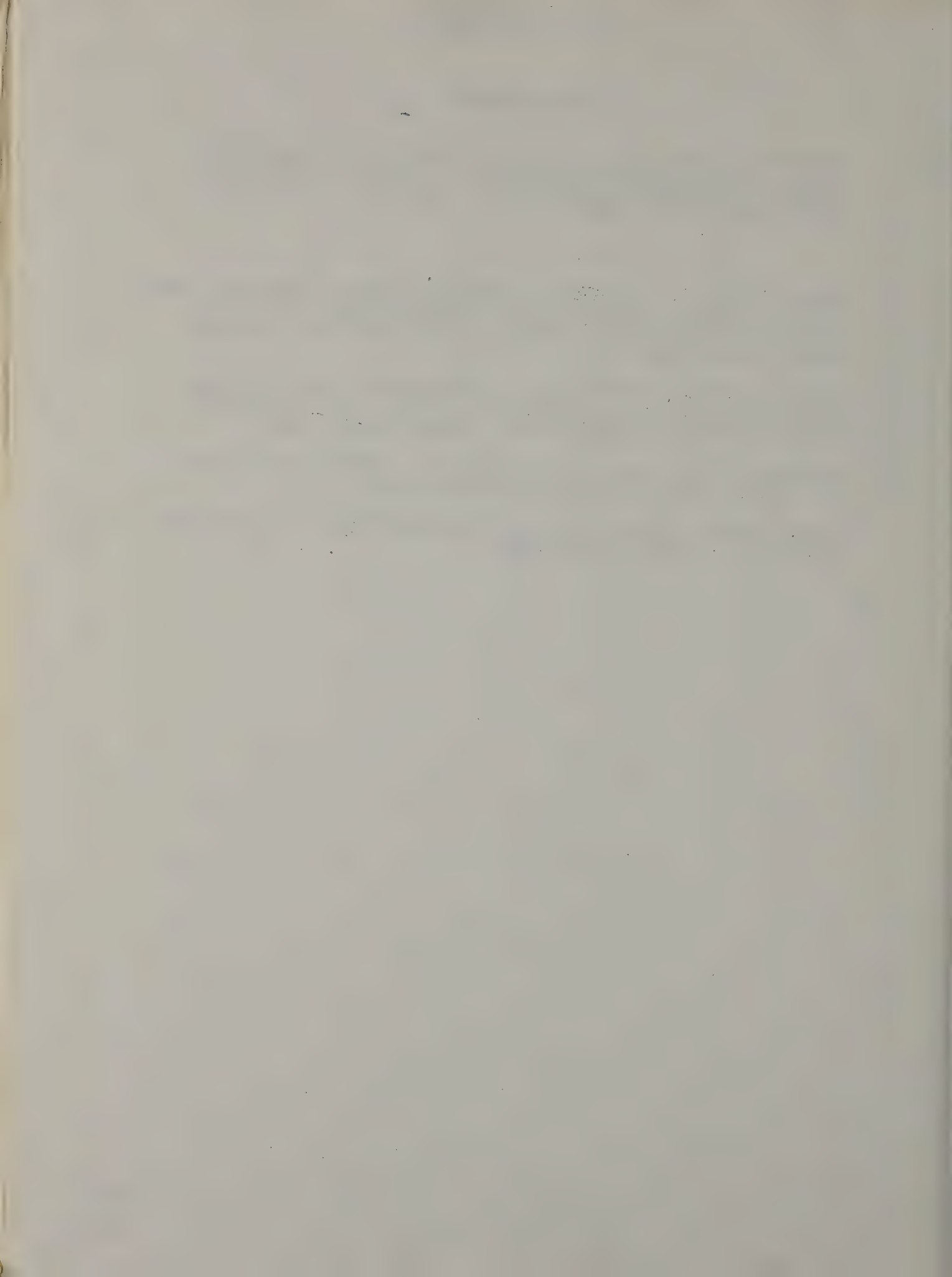
TABULATION OF DATA
OF
SELECTED SAMPLES

DELL HOLE #	DEPTH	LAB#	"N" Value	D ₅₀ mm	% - #200 Sieve	PLATE NO.
6	23.5-25.0	471	2	0.48	23	2
8	8.5-10.0	496	7	0.6	14	3
10	10-8 ⁵	577	-	0.5	22	8
10	33.5-35.0	567	6	0.5	25	9
10	36.5-38.0	569	4	0.6	11	10
14	10.0-11.5	581	8	0.6	12	12
18	5 ⁵ -7 ⁰	2720	15	0.5	21	13
18	14-15 ⁵	2725	6	0.85	15	14
18	20.5-22.0	2727	36	0.54	14	15
19	2.5-4.0	2735	5	0.48	16	16
19	12.5-14.0	2741	19	0.55	17	17
19	17.5-19.0	2744	13	0.46	23	18
20	4.0-5.5	2761	5	0.55	14	21
20	12.5-14.0	2766	9	0.51	22	21
20	17.5-19.0	2769	6	0.61	16	21
20	24.0-25.5	2773	13	0.7	11	19
20	25.5-27.0	2774	5	0.9	7	20



LIST OF REFERENCES

1. Seed, H.B., Idriss, I.M., Arango, I., "Evaluation of Liquefaction Potential Using Field Performance Data," Journal of Geotechnical Engineering, American Society of Civil Engineers, Vol. 109, No. 3, March, 1983, pp. 458 - 482.
2. Qamar, Anthony I., and Stickney, Michael C. "Montana Earthquakes, 1869 to 1979, Historical Seismicity and Earthquake Hazard," Memoir 51, Montana Bureau of Mines and Geology and the University of Montana, Butte, Montana, 1983.
3. Stickney, Michael and Bingler, E.C., "Earthquake Hazard Evaluation of the Helena Valley Area, Montana," Open File Report 83. Montana Bureau of Mines and Geology, Butte, Montana, August, 1981.
4. Department of the Army, Corps of Engineers, "Recommended Guidelines for Safety Inspection of Dams," Washington, D.C.
5. "Site Seismicity Evaluation, Mystic Lake Dam Project," International Engineering Company, August, 1983.



ote - Chessman aft 9-12-83
ofc - level for 1-2-84 is estimated

Chalis, Idaho
Note: Earthquake $\approx$ 8:10 AM 10-28-83

CITY OF HELENA
CHESSMAN DAM
JOB No. 83-575

Sheet 1 of 2

		PIEZOMETER NO. - DEPTH IN FEET																			
		20	30	40	50	60	70	80	90	100	110	120	130	140	150	160	170	180	190	200	
		P-1	P-2	P-3	P-4	P-5	P-6	P-7	P-8	P-9	P-10	P-11	P-12	P-13	P-14	P-15	P-16	P-17	P-18	P-19	P-20
1'	7-11-83	Dry	21 ¹	18 ²	15 ¹	19 ⁴	Dry	20 ³	19 ⁴	12 ⁷	14 ³	Dry	23 ¹⁰	20 ²	12 ⁰	14 ⁸	1 ⁹	2 ⁴	+3 ⁵	+2 ⁶	10 ³
1'	7-18-83	17 ⁶	20 ⁹	18 ⁴	15 ⁶	20 ⁵	Dry	23 ⁸	19 ⁸	12 ⁹	14 ¹¹	Dry	22 ⁶	19 ²	11 ⁹	16 ³	1 ⁹	1 ⁶	+3 ⁵	+2 ⁸	10 ¹
1'	7-25-83	19 ¹	21 ⁹	17 ¹	16 ³	19 ⁸	Dry	21 ⁵	18 ⁶	13 ¹¹	13 ⁶	Dry	20 ¹¹	19 ⁹	13 ⁷	14 ⁶	1 ⁹	1 ³	+2 ⁵	+1 ¹⁰	Broken Filled dirt
2'	8-1-83	19 ¹	21 ¹⁰	19 ³	19 ²	19 ⁴	Dry	23 ⁹	19 ¹¹	12 ⁷	13 ⁸	Dry	22 ⁷	20 ⁴	11 ⁸	15 ⁴	2 ¹	1 ⁶	+2 ⁹	+1 ¹¹	
3'	8-8-83	18 ⁸	21 ¹⁰	20 ¹⁰	16 ³	21 ²	Dry	23 ²	20 ⁴	12 ¹⁰	14 ⁷	Dry	22 ¹¹	20 ⁸	12 ⁶	15 ⁴	2 ⁶	1 ¹¹	+2 ⁶	+1 ⁶	
4'	8-15-83	19 ⁵	21 ⁴	20 ⁴	16 ⁸	20 ¹⁰	Dry	23 ⁵	20 ²	13 ²	14 ⁷	Dry	22 ⁶	20 ⁵	12 ⁷	15 ²	2 ³	2 ⁰	+2 ⁵	+1 ⁸	
5'	8-22-83	19 ⁴	22 ⁰	20 ⁹	17 ⁶	21 ³	Dry	23 ⁴	20 ⁶	13 ¹	14 ⁸	Dry	22 ³	20 ⁵	12 ⁷	17 ¹	2 ³	2 ³	+2 ⁴	+1 ⁷	
6'	8-29-83	19 ³	22 ¹	20 ⁸	16 ¹¹	21 ⁶	Dry	23 ¹	20 ⁴	12 ²	14 ⁹	Dry	22 ²	20 ²	12 ¹⁰	17 ³	2 ⁴	2 ⁵	+2 ³	+1 ⁸	
7'	9-5-83	20 ⁴	22 ⁶	21 ⁰	17 ²	21 ¹⁰	Dry	24 ⁰	19 ¹⁰	12 ⁶	15 ¹	Dry	22 ¹	20 ⁸	12 ⁰	17 ⁶	2 ³	1 ⁴	+2 ⁴	+1 ⁹	
8'	9-12-83	18 ⁸	22 ³	20 ¹	17 ⁵	21 ⁶	Dry	23 ⁸	20 ⁴	12 ⁹	15 ¹	Dry	23 ¹	2 ¹	12 ¹	16 ³	2 ⁴	1 ³	+2 ³	+1 ⁷	
9'	9-22-83	19 ⁴	22 ⁸	21 ⁶	18 ³	22 ⁴	Dry	24 ⁵	21 ⁵	12 ⁸	16 ²	Dry	24 ¹	22 ²	12 ¹¹	17 ³	3 ²	3 ¹	+2 ¹¹	+2 ⁴	
10'	9-26-83	19 ⁴	23 ¹	21 ⁹	17 ¹⁰	22 ²	Dry	24 ²	21 ¹	14 ²	15 ¹⁰	Dry	23 ⁴	23 ¹	13 ⁵	15 ⁹	2 ⁸	2 ⁵	+1 ⁹	+1 ¹	
11'	10-3-83	19 ⁴	23 ¹⁰	22 ⁶	18 ²	22 ⁹	Dry	24 ⁸	21 ⁵	14 ¹¹	16 ¹	Dry	24 ⁵	22 ⁰	13 ¹⁰	16 ¹⁰	2 ⁵	2 ⁸	+1 ⁹	+1 ⁹	
12'	10-10-83	19 ⁵	24 ²	22 ⁹	18 ³	23 ¹⁰	Dry	24 ⁹	22 ⁴	14 ¹⁰	17 ⁰	Dry	24 ⁶	22 ³	14 ¹	17 ⁵	2 ⁷	2 ⁴	+1 ⁵	+0 ⁶	
13'	10-17-83	19 ⁷	24 ⁵	24 ⁰	18 ⁸	25 ⁰	Dry	25 ⁷	22 ¹	16 ⁷	17 ²	Dry	24 ⁸	22 ⁶	14 ³	17 ²	2 ²	3 ⁵	0 ¹	0 ¹	11 ⁹
14'	10-24-83	Dry	24 ¹⁰	24 ⁴	19 ¹⁰	24 ⁶	Dry	25 ¹⁰	23 ⁷	17 ¹¹	18 ¹⁰	Dry	24 ¹⁰	23 ¹⁰	Dry	19 ⁴	3 ⁵	3 ⁶	0 ¹	0 ²	12 ⁶
15'	10-31-83	Dry	26 ¹	23 ⁹	Dry	24 ²	Dry	25 ¹¹	23 ¹⁰	17 ⁰	18 ¹¹	Dry	25 ²	23 ¹⁰	Dry	19 ¹	3 ⁰	3 ⁶	0 ¹	0 ²	12 ⁷
16'	11-7-83	Dry	25 ⁴	24 ³	Dry	24 ⁷	Dry	26 ¹	27 ⁴	17 ¹¹	18 ¹⁰	Dry	27 ⁵	24 ¹	Dry	19 ²	2 ⁰	3 ⁶	0 ²	0 ²	12 ⁵
17'	11-14-83	Dry	26 ²	24 ²	Dry	25 ⁰	Dry	26 ²	24 ³	17 ⁵	18 ¹⁰	Dry	25 ⁴	24 ²	Dry	20 ⁰	2 ²	3 ⁶	0 ²	0 ²	12 ²
18'	11-18-83	Dry	26 ²	24 ¹	Dry	25 ¹	Dry	26 ²	24 ³	17 ⁶	18 ¹¹	Dry	25 ³	24 ⁷	Dry	20 ⁰	2 ²	3 ⁶	0 ²	0 ²	13 ²
19'	11-27-83	Dry	26 ²	24 ⁷	Dry	25 ²	Dry	26 ⁷	24 ⁴	17 ⁵	19 ⁴	Dry	25 ⁹	24 ⁷	Dry	20 ⁰	3 ³	3 ⁷	Froze	0 ⁴	13 ³
20'	12-1-83	Dry	26 ²	24 ⁷	Dry	25 ²	Dry	26 ⁹	24 ⁶	18 ⁰	10 ⁴	Dry	25 ¹⁰	24 ⁷	Dry	20 ⁰	3 ⁴	3 ⁷	Froze	Froze	13 ²
21'	12-14-83	Dry	26 ⁵	24 ²	Dry	25 ³	Dry	26 ⁹	24 ⁹	18 ⁴	19 ⁴	Dry	26 ²	24 ⁶	Dry	20 ⁰	3 ⁴	3 ⁷	Froze	Froze	13 ⁴
22'	1-2-84	Dry	26 ⁵	25 ³	Dry	25 ⁴	Dry	26 ⁹	25 ⁵	19 ⁶	20 ²	Dry	26 ²	24 ⁸	Dry	20 ⁰	3 ⁴	3 ⁹	Froze	Froze	13 ⁵
23'	1-9-84	Dry	26 ⁵	25 ⁴	Dry	25 ⁵	Dry	26 ¹¹	25 ⁵	19 ³	20 ¹¹	Dry	26 ²	24 ⁸	Dry	20 ⁰	3 ⁴	3 ¹⁰	Froze	Froze	13 ¹⁰
24'	1-30-84	Dry	26 ⁵	25 ⁴	Dry	25 ⁶	Dry	27 ¹	25 ⁶	19 ⁸	20 ⁰	Dry	26 ⁴	24 ⁹	Dry	21 ⁰	3 ⁶	3 ¹¹	Froze	Froze	13 ¹¹
25'	2-9-84	Dry	26 ⁵	25 ⁴	Dry	26 ⁰	Dry	27 ⁵	25 ²	19 ¹⁰	21 ⁶	Dry	26 ⁶	25 ³	Dry	21 ⁰	3 ⁷	3 ⁴	Froze	Froze	Froze
26'	2-21-84	Dry	26 ⁴	25 ⁴	Dry	26 ⁰	Dry	27 ⁶	25 ⁶	Dry	21 ⁶	Dry	26 ⁷	25 ⁹	Dry	21 ²	3 ⁷	3 ⁷	Froze	Froze	Froze

CHESSMAN DAM

PIEZOMETER NO. DEPTH IN FEET

RESER- VOIR		20'		30'		40'		20'		30'		20'		30'		53'		20'		30'		30'		60'		10'	
ELEV.	DATE	P-1	P-2	P-3	P-4	P-5	P-6	P-7	P-8	P-9	P-10	P-11	P-12	P-13	P-14	P-15	P-16	P-17	P-18	P-19	P-20						
7'-9"	4-24-84	Dry	26 ⁷	26 ⁵	Dry	26 ¹⁰	Dry	28 ¹¹	26 ⁶	Dry	24 ¹	Dry	28 ⁵	27 ²	Dry	23 ⁵	3 ⁸	3 ⁶	0 ⁸	Frz							
8'	7-3-84	"	27 ²	26 ⁸	"	27 ⁹	"	28 ⁵	27 ⁴	"	25 ³	"	28 ¹¹	27 ¹	"	25 ³	7 ¹⁰	4 ⁶	10	13	13 ¹¹						



H. Bolton Seed, Inc.

623 CROSSRIDGE TERRACE, ORINDA, CALIFORNIA 94563

(415) 254-3036

November 18, 1983

Walter Jones
Northern Engineering and Testing, Inc.
600 South 25th Street
Box 30615
Billings, Montana 59107

Dear Mr. Jones,

In response to your request I have reviewed the various information you sent me on September 26 concerning the seismic safety evaluation of Chessman Dam in Helena, Montana. This information included:

1. Logs of the test borings made at the dam.
2. Results of laboratory tests on the samples.
3. Correspondence relating your preliminary findings.
4. The Phase I dam safety inspection report.
5. Plan view locating the borings, and typical cross section.
6. Piezometer monitoring results.
7. Typical embankment cross sections perpendicular to the centerline.
8. Background report on seismicity in the area.

On the basis of this review, which indicated a great deal of clayey sands and sandy clays in the cross-section and foundation of the embankment, I informed you that there was a reasonably good possibility that the dam would not be susceptible to liquefaction in the event of strong earthquake shaking but I suggested that in order to clarify this matter, several additional borings be made in the downstream section of the embankment and special care be taken to observe the nature of the soils disclosed by these borings. I also suggested that when this data became available, detailed cross-sections through the embankment and foundation be prepared to permit a more detailed study of the embankment and foundation conditions.

At a meeting in my office on October 26, you presented preliminary plots showing the results of these studies and while it was apparent that the sections contained a great deal of non-liquefiable clayey soils, there was also found to be a layer of relatively loose clean sand underlying a portion of the embankment. This layer appears to have characteristics which could cause it to liquefy under the maximum earthquake ground shaking which could occur in the vicinity of the dam.

Accordingly we tentatively concluded that some remedial work would be necessary to provide an adequately stable section from the point of view of earthquake stability and that you would explore the extent of this work in conjunction with other measures required for improving the condition of the embankment.

I understand that you will send me a draft report for final review when it is completed.

Sincerely yours,

A handwritten signature in cursive script that reads "H. Bolton Seed". The signature is written in dark ink and is positioned above the printed name.

H. Bolton Seed

Commissioners

Russell J. Ritter, Mayor
Rayleen Beaton
Michael J. DaSilva
Joan A. Duncan
Blake Wordal

Robert A. Erickson
City Manager



City-County Admin. Bldg.
316 North Park
Helena, MT 59623
Phone 406/442-9920

September 13, 1984

hze

Mr. Harold Eagle
Morrison-Maierle, Inc.
P. O. Box 6147
Helena, Montana 59604

Re: TV Inspection of Chessman Dam

Dear Harold:

Transmitted herewith is a copy of a memo and TV log dated August 16, 1984, from Elmer Cole to myself regarding inspection of the pipe at Chessman Dam with our TV camera. As you will recall, you and members of your firm reviewed the videotape of the inspection, and we discussed with Elmer his visual observations of the pipe since he was in it all the way to the 16" section. It appears from the videotaping and his report that there has been no apparent lateral or vertical movement of the pipe since its installation in the early 1900s. There appear, however, to be a few joints that would need to be grouted in any rehabilitation project on the dam. This would be of minimal cost, would involve one man's labor for probably a day or two and could be completed by City of Helena personnel.

In my opinion due to the excellent condition of this pipe, should Chessman be used at a later date, replacement of this line as part of the rehabilitation project would not be necessary. The City would save roughly \$100,000 to \$125,000 in costs that would have been required should replacement of this line been necessary. This data is for your files and would not necessarily have to be included in the Chessman report you are presently preparing; however, could be attached as an appendix if you so desire.

Sincerely,

A handwritten signature in dark ink, appearing to read "Richard A. Nisbet". The signature is fluid and cursive, with a large, sweeping "R" and "N".

RICHARD A. NISBET, P.E.
Director of Public Works

RAN/nn

Enclosures

cc: Robert A. Erickson, City Manager
Elmer Cole, Sewer Maintenance Foreman
Dickert, Willett, Griffiths

MEMORANDUM

J.
RECEIVED

TO: Richard Nisbet, Director of Public Works

AUG 22 1984

FROM: Elmer Cole, Sewer Maintenance Foreman

PUBLIC WORKS

DATE: August 16, 1984

SUBJ: Chessman Stability Analysis 82-83

Dick after spending the better part of the day up at Chessman Reservoir setting up and T.V. ing the pipe line, we report the following findings.

There are two different pipes involved, one is a 24" concrete pipe that was cast in place and poured in two sections. This pipe is not round in diameter. In fact we could have used a camera skid between 24" and 30". The distance of this section was 82 feet.

The second pipe is 16" steel or cast iron pipe. It has large amounts of scale on the sides, but in general it is in good shape. The length of this pipe is 100 feet, at a distance of 15 feet into this section is the valve control. The distance of this pipe from the outlet below the dam to the end of the pipe at the drop inlet in the lake itself is 192 feet. We first set the camera skids up to size 18 inch and tried to T.V. the line, this didn't work because the camera was under water due to the outlet of the pipe being higher than the flow out of the line causing this section to back up. We then changed to 24" skids and started up the line using the video tape recorder. At a distance of 32 feet there was a five gallon plastic bucket in the pipe and this had to be removed by crawling in the pipe and dragging it out. The line was then T.V.'d to a distance of 82 feet where the pipe necks down to 16". Because of debris at the lower end of the system, there was about 8" of water in the pipe even with the valve closed off. This could be fixed by using the backhoe on the outlet. Because of the thickness of scale on the sides of the pipe we used 12" skids to T.V. the remaining distance. The camera had to be carried into the 24" pipe because of the rough joints and the amount of water. We pushed the camera into the 16" pipe and at 97' the valve was located. From here to the end of the line other than scale the pipe was in good condition. The camera was pulled to the end of the line which is 192 feet. Please see video tape for further details. At one point there is a broken section of pipe which has a large crack or opening along the top quadrant and a 2" wide crack at the bottom. The distance to this joint is 62'. See attachments for drawing and T.V. log.

Respectfully submitted,

Elmer Cole

Elmer Cole
Sewer Maintenance Foreman

cc Ben Spady, Wastewater Superintendent
Charlie Dickert, Water Superintendent

City of Helena, Montana

Chessman Reservoir

Camera pulled
to here (stop)

drop Inlet

← Roadway →

□ - T.V. VAN

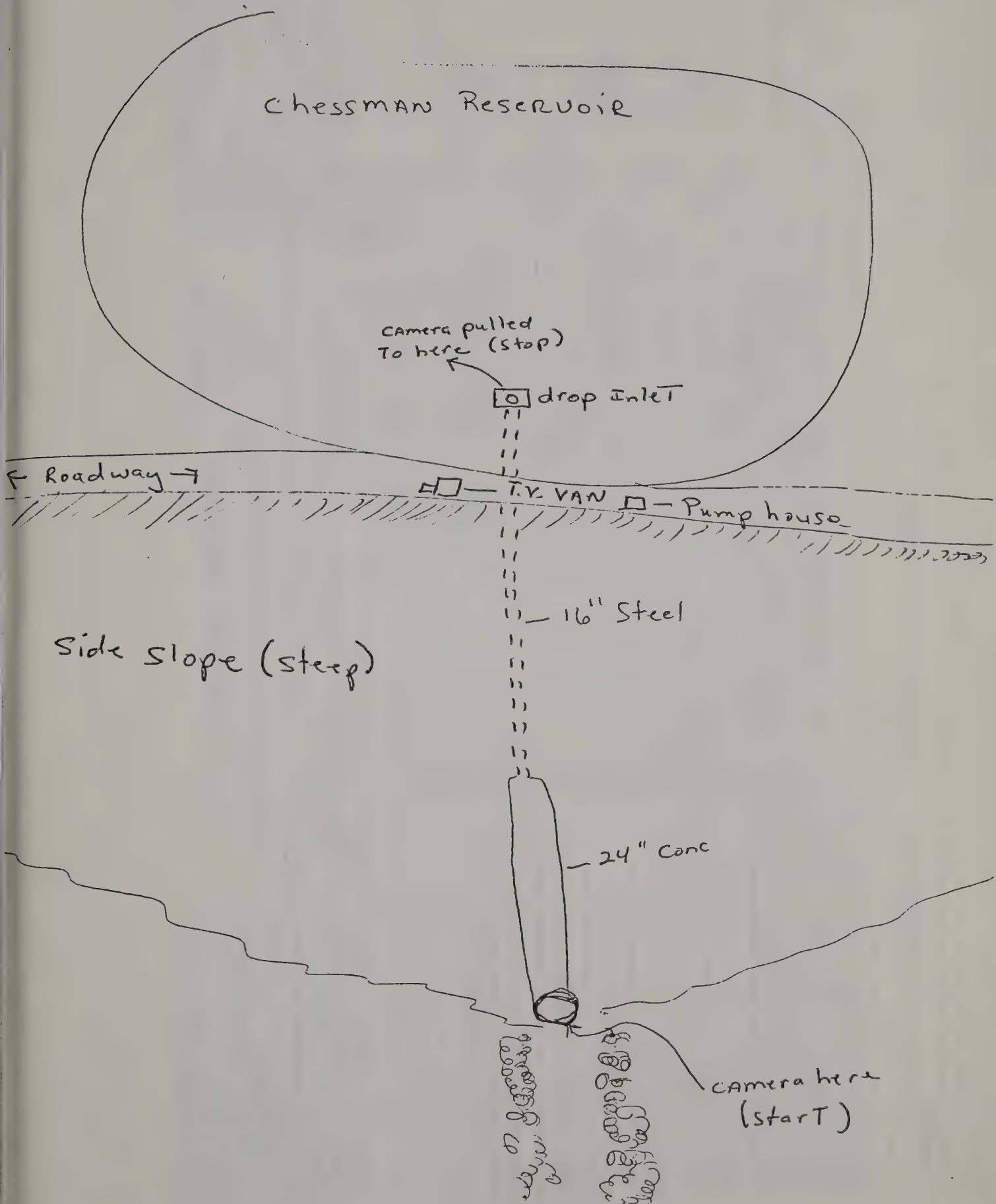
□ - Pump house

16" Steel

Side slope (steep)

24" Conc

Camera here
(start)



PIPE SIZE 24" CONC. 16" STEEL
 TYPE OF PIPE
 TYPE OF JOINT mortar joint
 SECTION LENGTH
 MH DEPTHS

CITY OF HELENA MONTANA
 SANITARY SEWERS WORK SHEET

quadrant nos.
 4 1
 3 2

DATE INSPECTED 8-15-84
 AREA BOOK
 KEY SHEET
 STREET
 MANHOLE NO. TONO.

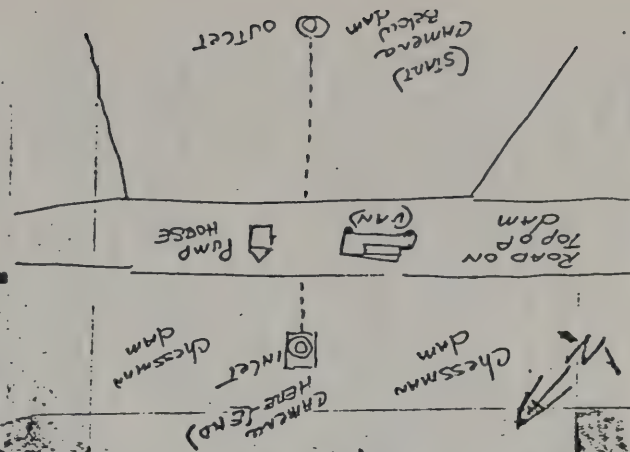
INSPECTION CREW: HORNER, Cole, Halbert

LOCATION Chessman Dam

NEW OR OLD

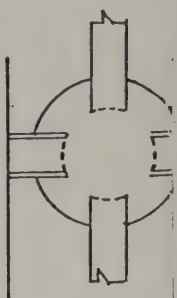
DISTANCE READING	SERVICE TAPS	SERVICE WYES	PROTRUDING SERVICES	QUADRANT				PHOTO NO	TAPE	REMARKS	VIEWING	DIRECTION
				1	2	3	4				↑	
9'						X	X			Start at Cullet side 24" CRACK IN PIPE NOT REAL BAD		
15'										Joint in pipe, bottom WAS CONC, top looks like 1x4" LUMBER WAS USED FOR top of pipe.		
26'				X				X		Joint here, has bare line CRACK IN QUAD 4+1		
33'										Joint in pipe, top looks OK but ALL JOINTS ARE WIDE about fist size on bottom of joints No mortar was used here.		
44'										Joint in pipe O.K on top wide on the bottom.		

PIPE CONDITION 24" CONC. has some cracks in pipe. Also all joints are manhole condition wide on the bottom, perhaps not mortared, or washed away. GRADE IF KNOWN 16" steel pipe is in good shape, other than scaled up 2" REMARKS on each side of pipe.



DIRECTION OF MEASUREMENT ———— FROM CENTER OF MANHOLE

MANHOLE NO. ———— MANHOLE NO. ————

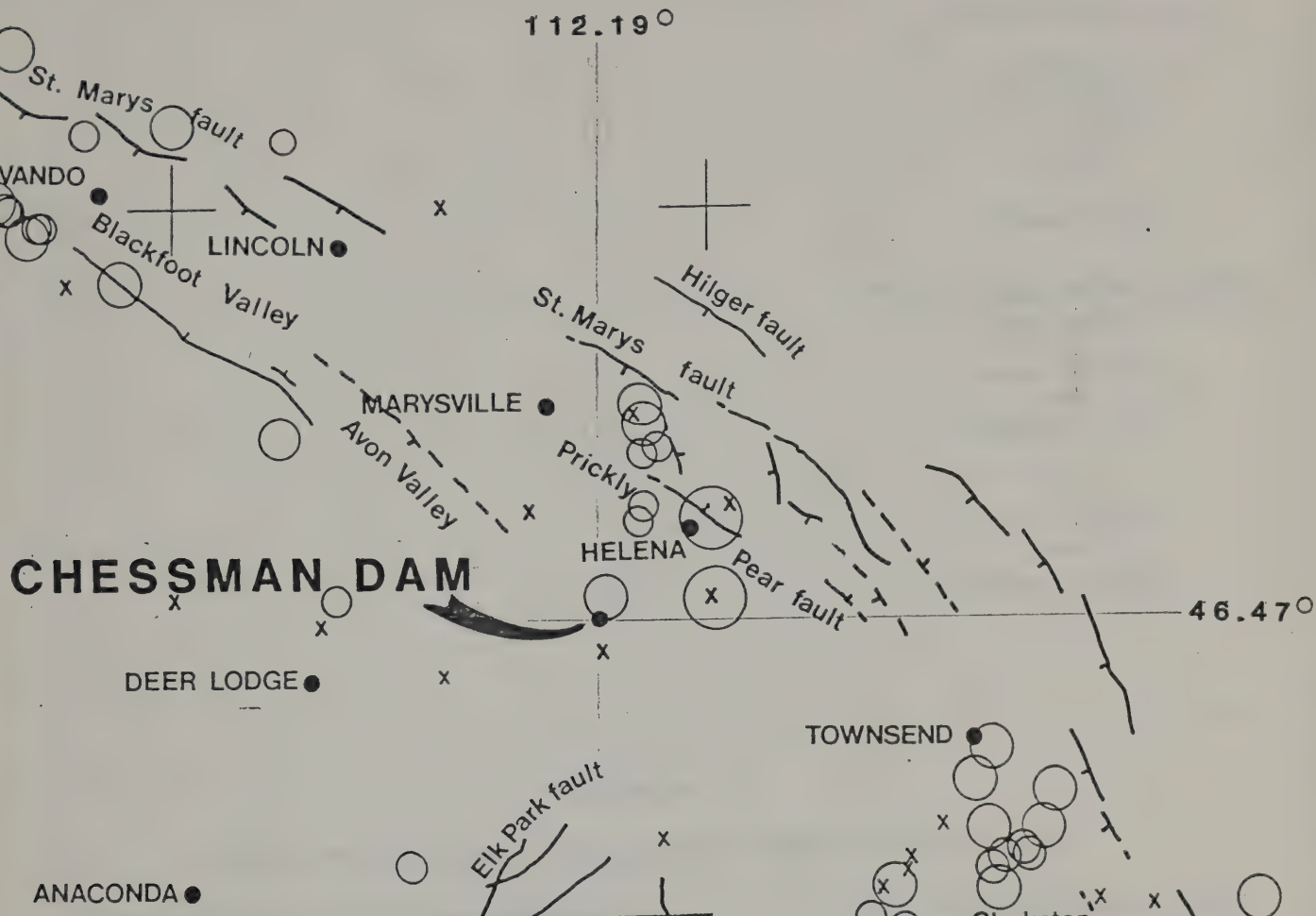


(41 / 32)

QUADRANTS

T.V. INSPECTION

CREW LEADER AND DATE	REMARKS	LEAK NO	DISTANCE READING	QUADRANT 1 2 3 4	PHOTO NO	REMARKS
50'						Joint here again looks good on top but
51'				X X		wide on the bottom
56'				/		CRACK IN QUAD 3 & 4
60'						Joint in pipe O.K. at top, wide at the bottom.
62'						" " " "
69'						BROKEN JOINT HERE
75'						Joint here O.K. at top, wide at the bottom.
↓						UNDER WATER
80'						↓
82'						At this point pipe changes to different
97'						diameter pipe 16" steel
101'						knife valve here looks O.K.
113'						Joint in pipe looks good. Pipe badly
126'						OPERATED
137'						Joint looks good
142'						" " "
147'						" " "
192'						END of line center of inlet pipe



Reprinted from:
**Historical Earthquakes in Montana
 and Vicinity, 1869-1979**

MEMOIR 51

**Qamar and Stickney, 1983
 Sheet 1 (of 3)**

Explanation

Epicenter Magnitude

○ 7

○ 5

○ 3

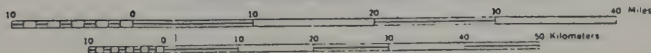
X not computed

— Normal fault—barb
 on downthrown side;
 dashed where inferred.

— Strike-slip fault.

Scale 1:1,000,000

1 inch equals approximately 16 miles



**CHESSMAN DAM
 REHABILITATION INVESTIGATION**



MORRISON-MAIERLE, INC.
 CONSULTING ENGINEERS

Reprinted from:



Memoir 51

MONTANA EARTHQUAKES, 1869-1979

Historical Seismicity
and
Earthquake Hazard

by

Anthony I. Qamar
and

Michael C. Stickney

23

Appendix C

Principal earthquakes in Montana, 1869-1979—listed by region.

Historical earthquakes in Montana are given for each region shown in Figure 2. Sources of information include **Appendix B**; the hypocenter data file of the National Oceanic and Atmospheric Administration; and the U.S. Department of Commerce annual publication, *United States Earthquakes*. Earthquakes with instrumentally determined epicenters (**Sheet 1, back pocket**) and earthquakes only reported *felt* are tabulated (Figure 3).

Explanation of tables:

- (1) **Date:** Given as day-month-year. Zero indicates uncertain day (or month).
- (2) **Time:** Origin time of earthquake in Mountain Standard Time (MST). Add seven hours to obtain universal time. Only hour and minute given; 00.00 indicates uncertain time origin.
- (3) **Lat, Long:** Latitude and longitude in degrees. For example, 48.50, 114.75 means 48°30'N, 114°45'W. An asterisk beside latitude and longitude indicates an instrumental (more accu-

(4) **Mag, C:**

rate) location. Coordinates without asterisks are estimated epicenters based on *felt* reports.

MAG gives earthquake magnitude. Zero means no magnitude computed. The number given in MAG can be from various magnitude scales as follows:

C	Magnitude scale for MAG
0	No magnitude given
1	M_s , surface wave magnitude
2	m_b , body wave magnitude
3	Based on another magnitude scale
4	M_L , Local or Richter magnitude scale

(5) **In:**

Maximum Modified Mercalli Intensity. Zero indicates no intensity estimated.

(6) **Area:**

Area over which earthquake was *felt* in thousands of square kilometers. For example, 27. implies a *felt* area of 27,000 km².

(7) **S:**

1, 2 or 3 indicates epicenter in Montana, Idaho or Wyoming. (If zero, state is not specified.)

Region 4—Helena-Ovando

D A T E	T I M E	L A T	L O N G	M A G	C	I N	A R E A	S	C O M M E N T S
22/05 1869	00:00	46.60	112.03	0.0	0	6	0. 1	HELENA (MORNING)	
10/12 1872	16:30	46.40	112.50	0.0	0	6	0. 1	DEER LODGE, MISSOULA, HELENA, SHOCKS ALSO DEC11(0230,0655), DEC12(0700	
08/03 1877	02:00	46.60	112.03	0.0	0	0	0. 1	HELENA, LIGHT SHOCK	
04/05 1883	11:45	46.60	112.03	0.0	0	0	0. 1	HELENA	
11/11 1885	09:15	46.60	112.03	0.0	0	3	0. 1	HELENA	
24/03 1893	06:15	46.60	112.03	0.0	0	0	0. 1	HELENA	
26/01 1895	05:00	46.60	112.03	0.0	0	0	0. 1	HELENA	
05/06 1897	05:22	46.60	112.03	0.0	0	0	0. 1	S.OF HELENA. FELT BUTTE, BOZEMAN, DEERLODGE, ANACONDA	
19/03 1898	06:00	46.77	112.27	0.0	0	0	0. 1	MARYSVILLE	
15/03 1903	10:57	46.60	112.03	0.0	0	6	0. 1	HELENA, TOWNSEND, WINSTON, WHITE SULFUR SPRINGS (OR INTENSITY 05)	
23/11 1905	16:30	46.60	112.03	0.0	0	4	0. 1	HELENA	
11/10 1909	18:25	46.60	112.03	0.0	0	0	0. 1	HELENA	
16/08 1920	11:03	46.60	112.03	0.0	0	3	0. 1	HELENA	
18/12 1922	09:27	46.60	112.03	0.0	0	4	0. 1	HELENA	
20/02 1923	16:14	46.60	112.03	0.0	0	4	0. 1	HELENA ?	
22/05 1923	01:07	46.60	112.03	0.0	0	4	0. 1	HELENA ?	
31/05 1925	00:00	46.60	112.03	0.0	0	3	0. 1	HELENA ?	
30/09 1925	02:30	46.60	112.03	0.0	0	0	0. 1	HELENA (LASTED 6 SECONDS)(REALLY CLARKSTON?)	
24/03 1926	00:00	46.60	112.03	0.0	0	0	0. 1	FELT AT HELENA, KENWOOD	
29/02 1928	15:38	46.50	112.00	0.0	0	5	0. 1	HELENA, STRONG SHOCK, GENERALLY FELT IN WESTERN MONTANA(POSS. I=4)	
24/07 1928	11:55	46.60	112.03	0.0	0	2	0. 1	HELENA	
31/05 1929	05:30	46.50	112.00	0.0	0	2	0. 1	HELENA (ALSO AT 0533)	
16/03 1930	05:54	46.50	112.00	0.0	0	6	0. 1	HELENA (ALSO AT 0613 OR 0623)	
08/06 1931	06:26	46.80	112.50	0.0	0	2	0. 1	HELENA (ALSO AT 0750)	
03/11 1931	14:25	46.58	112.00	0.0	0	2	0. 1	HELENA	
03/10 1932	22:19	46.60	112.03	0.0	0	2	0. 1	HELENA	
11/10 1932	16:05	46.60	112.03	0.0	0	2	0. 1	HELENA	
04/05 1933	11:54	46.60	112.03	0.0	0	3	0. 1	HELENA	
10/06 1933	22:59	46.60	112.03	0.0	0	5	0. 1	HELENA	
03/08 1934	19:20	47.27	111.68	0.0	0	4	0. 1	CASCADE	
03/10 1935	19:47	46.60	112.03	0.0	0	5	0. 1	HELENA	
03/10 1935	20:21	46.60	112.03	0.0	0	3	0. 1	HELENA	
07/10 1935	12:30	47.00	111.90	0.0	0	6	0. 1	CRAIG	
12/10 1935	00:51	46.62*	111.97*	0.0	0	7	181. 1	HELENA (SCRATCH GRAVEL)	
15/10 1935	13:30	46.60	112.02	0.0	0	6	0. 1	HELENA (ALSO AT 1402, 1421)	
18/10 1935	19:40	46.30	112.02	0.0	0	0	0. 1	JEFFERSON CITY	
18/10 1935	21:48	46.60*	112.00*	6.3	3	8	596. 1	HELENA	
18/10 1935	22:30	46.30	112.02	0.0	0	2	0. 1	JEFFERSON CITY	
19/10 1935	03:31	46.43	111.56	0.0	0	0	0. 1	ALHAMBRA (ALSO AT 0416)	
19/10 1935	03:31	46.60	112.03	0.0	0	0	0. 1	HELENA (ALSO AT 0416, MANY AFTERSHOCKS)	

Region 4—Helena-Ovando (continued)

D A T E	T I M E	L A T	L O N G	M A G	C	I N	A R E A	S	C O M M E N T S
21/10 1935	03:33	46.60	112.03	0.0	0	5	0. 1	HELENA	
21/10 1935	06:32	46.60	112.03	0.0	0	0	0. 1	HELENA	
23/10 1935	10:40	46.60	112.03	0.0	0	5	0. 1	HELENA	
25/10 1935	16:00	46.60	112.03	0.0	0	0	0. 1	HELENA (ALSO AT 1730, RUMBLING SOUND)	
27/10 1935	12:00	46.62	111.97	0.0	0	6	0. 1	HELENA	
31/10 1935	08:20	46.90	112.40	0.0	0	4	0. 1	WILBORN (AM OR PM)	
31/10 1935	11:37	46.50*	112.00*	6.0	3	8	363. 1	HELENA	
02/11 1935	10:42	46.60	112.03	0.0	0	5	0. 1	HELENA (DURATION 3 SEC, AFTERSHOCK)	
04/11 1935	04:23	46.60	112.03	0.0	0	6	0. 1	HELENA	
04/11 1935	05:42	46.60	112.03	0.0	0	5	0. 1	HELENA	
06/11 1935	08:23	46.60	112.03	0.0	0	5	0. 1	HELENA (DURATION 3 SEC, AFTERSHOCK)	
21/11 1935	20:58	46.62	111.97	0.0	0	6	3. 1	HELENA, BOZEMAN, BUTTE, GREAT FALLS, VIRGINIA CITY	
28/11 1935	07:41	46.60*	112.00*	0.0	0	6	233. 1	HELENA (TWO SHOCKS HORIZONTAL, ONE TWISTING)	
13/02 1936	16:55	46.60	112.03	0.0	0	5	0. 1	HELENA, BUTTE, GREAT FALLS, HAMILTON (OR INTENSITY 06)	
13/02 1936	17:30	46.60	112.03	0.0	0	6	0. 1	HELENA, LEWISTOWN, LIVINGSTON, MISSOULA	
25/03 1936	00:00	46.77	112.27	0.0	0	5	0. 1	MARYSVILLE (MANY EGS)	
13/05 1936	07:06	46.60	112.03	0.0	0	4	0. 1	HELENA	
21/05 1936	19:19	46.60	112.03	0.0	0	4	0. 1	HELENA	
03/06 1936	01:31	46.60	112.03	0.0	0	4	0. 1	HELENA	
11/06 1936	16:13	46.60	112.03	0.0	0	5	0. 1	HELENA, BIRDSEYE (OR INTENSITY 06)	
12/06 1936	04:40	46.60	112.03	0.0	0	5	0. 1	HELENA	
14/06 1936	08:03	46.60	112.03	0.0	0	4	0. 1	HELENA	
21/06 1936	11:15	46.95	112.57	0.0	0	2	0. 1	7 MI E. OF LINCOLN	
22/06 1936	20:30	46.77	112.27	0.0	0	5	0. 1	MARYSVILLE	
29/06 1936	04:27	46.60	112.03	0.0	0	5	0. 1	HELENA, BUTTE, GREAT FALLS	
04/09 1936	02:00	46.65	112.06	0.0	0	3	0. 1	5.5 MI N.N.W. OF HELENA (AFTERSHOCKS)	
09/10 1936	03:21	46.60	112.03	0.0	0	4	0. 1	HELENA	
31/10 1936	22:00	46.60	112.03	0.0	0	3	0. 1	KENWOOD (2200 TO 2300)	
00/10 1936	00:00	46.77	112.27	0.0	0	0	0. 1	MARYSVILLE (TREMORS DAILY)	
13/12 1936	23:53	46.60	112.03	0.0	0	4	0. 1	HELENA	
23/05 1937	01:16	46.60	112.03	0.0	0	4	0. 1	HELENA	
06/10 1937	23:43	46.60	112.03	0.0	0	3	0. 1	HELENA	
15/10 1937	23:56	46.60	112.03	0.0	0	3	0. 1	HELENA	
19/11 1937	11:30	46.60	112.03	0.0	0	3	0. 1	HELENA	
27/11 1937	19:11	46.60	112.03	0.0	0	3	0. 1	HELENA	
21/01 1938	12:53	46.60	112.03	0.0	0	3	0. 1	HELENA	
24/04 1938	11:23	46.60	112.03	0.0	0	4	0. 1	HELENA	
04/08 1938	00:07	46.60	112.03	0.0	0	3	0. 1	HELENA	
23/08 1938	04:00	46.40	112.74	0.0	0	2	0. 1	DEERLODGE	
28/12 1938	15:53	46.60	112.03	0.0	0	3	0. 1	HELENA	
10/06 1939	10:35	46.75	112.17	0.0	0	4	0. 1	SILVER CITY	
17/11 1939	23:38	46.60	112.03	0.0	0	4	0. 1	HELENA	
19/08 1940	00:09	46.60	112.03	0.0	0	4	0. 1	HELENA	
23/12 1940	14:50	46.50*	112.50*	0.0	0	6	18. 1	HELENA. ALSO FELT IN MISSOULA, GT FALLS, CARDWELL	
25/12 1940	20:30	46.86	112.96	0.0	0	3	0. 1	HELMVILLE	
03/03 1942	14:12	46.60	112.03	0.0	0	3	0. 1	HELENA	
25/06 1943	16:20	46.60	112.03	0.0	0	2	0. 1	HELENA (ALSO AT 1758)	
12/12 1943	05:41	46.60	112.03	0.0	0	3	0. 1	HELENA	
06/06 1944	16:18	46.60	112.03	0.0	0	3	0. 1	HELENA	
05/09 1944	11:20	46.60	112.03	0.0	0	3	0. 1	HELENA	
12/09 1944	02:45	46.60	112.03	0.0	0	3	0. 1	HELENA	
29/11 1944	08:00	46.60	112.03	0.0	0	3	0. 1	HELENA	
15/01 1945	05:10	46.60	112.03	0.0	0	3	0. 1	HELENA	
20/01 1945	11:18	46.60	112.03	0.0	0	3	0. 1	HELENA	
23/01 1945	04:01	46.60	112.03	0.0	0	2	0. 1	HELENA	
02/02 1945	05:03	46.60	112.03	0.0	0	3	0. 1	HELENA (DURATION 4 SEC)	
14/02 1945	17:49	46.60	112.03	0.0	0	2	0. 1	HELENA	
01/03 1945	09:50	46.60	112.03	0.0	0	2	0. 1	HELENA (ALSO AT 1003, 1136)	
02/03 1945	10:43	46.60	112.03	0.0	0	2	0. 1	HELENA	
02/03 1945	19:37	46.60	112.03	0.0	0	4	0. 1	HELENA	
02/03 1945	19:38	46.60	112.03	0.0	0	3	0. 1	HELENA	
02/03 1945	19:43	46.60	112.03	0.0	0	2	0. 1	HELENA	
03/03 1945	01:43	46.60	112.03	0.0	0	4	0. 1	HELENA (ALSO AT 0205, 0210, 0214)	
04/03 1945	06:19	46.60	112.03	0.0	0	2	0. 1	HELENA (ALSO AT 1719)	
05/03 1945	10:47	46.60	112.03	0.0	0	3	0. 1	HELENA	
05/03 1945	11:10	46.60	112.03	0.0	0	2	0. 1	HELENA (ALSO AT 1745, 1757)	
06/03 1945	09:54	46.60	112.03	0.0	0	2	0. 1	HELENA	
18/03 1945	01:58	46.60	112.03	0.0	0	3	0. 1	HELENA (TWELVE SHOCKS 0158 THROUGH 0645)	
26/03 1945	22:13	46.60	112.03	0.0	0	3	0. 1	HELENA	

Region 4—Helena-Ovando (continued)

D A T E	T I M E	L A T	L O N G	M A G	C	I N	A R E A	S	C O M M E N T S
01/04	1945	01:15	46.60	112.03	0.0	0	3	0. 1	HELENA
01/04	1945	01:40	46.60	112.03	0.0	0	2	0. 1	HELENA
03/04	1945	22:45	46.60	112.03	0.0	0	2	0. 1	HELENA
04/04	1945	00:01	46.60	112.03	0.0	0	3	0. 1	HELENA (ALSO AT 0845, 1141)
05/04	1945	02:25	46.60	112.03	0.0	0	4	0. 1	HELENA
05/04	1945	17:25	46.60	112.03	0.0	0	3	0. 1	HELENA
08/04	1945	17:23	46.60	112.03	0.0	0	2	0. 1	HELENA
09/04	1945	16:24	46.60	112.03	0.0	0	2	0. 1	HELENA
10/04	1945	17:10	46.60	112.03	0.0	0	5	0. 1	HELENA (1710 THROUGH 1940)
11/04	1945	06:29	46.60	112.03	0.0	0	2	0. 1	HELENA (ALSO AT 1001)
12/04	1945	23:50	46.60	112.03	0.0	0	3	0. 1	HELENA (SEVERAL AFTERSHOCKS)
14/04	1945	06:54	46.60	112.03	0.0	0	3	0. 1	HELENA
15/04	1945	11:58	46.60	112.03	0.0	0	2	0. 1	HELENA
16/04	1945	11:20	46.60	112.03	0.0	0	2	0. 1	HELENA (ALSO AT 1210)
17/04	1945	00:14	46.60	112.03	0.0	0	3	0. 1	HELENA
17/04	1945	21:00	46.60	112.03	0.0	0	2	0. 1	HELENA (ALSO AT 2200)
19/04	1945	18:25	46.60	112.03	0.0	0	2	0. 1	HELENA
20/04	1945	03:45	46.60	112.03	0.0	0	2	0. 1	HELENA (ALSO AT 2114)
21/04	1945	06:28	46.60	112.03	0.0	0	2	0. 1	HELENA
06/05	1945	11:43	46.60	112.03	0.0	0	3	0. 1	HELENA
09/05	1945	16:16	46.60	112.03	0.0	0	2	0. 1	HELENA (ALSO AT 1916, 2104)
11/05	1945	18:33	46.60	112.03	0.0	0	2	0. 1	HELENA (ALSO AT 1836, 1840, 1916)
01/06	1945	09:54	46.60*	112.00*	0.0	0	5	16. 1	HELENA
01/06	1945	10:19	46.60	112.03	0.0	0	4	0. 1	HELENA (1019 THROUGH 1326, AFTERSHOCKS)
02/06	1945	15:36	46.60	112.03	0.0	0	2	0. 1	HELENA (1536 THROUGH 1539)
03/06	1945	14:46	46.60	112.03	0.0	0	3	0. 1	HELENA
03/06	1945	15:05	46.60	112.03	0.0	0	2	0. 1	HELENA
05/06	1945	08:12	46.60	112.03	0.0	0	2	0. 1	HELENA (ALSO AT 1752)
06/06	1945	14:00	46.60	112.03	0.0	0	3	0. 1	HELENA
10/06	1945	15:47	46.60	112.03	0.0	0	2	0. 1	HELENA (ALSO AT 1625)
11/06	1945	11:45	46.60	112.03	0.0	0	2	0. 1	HELENA (ALSO AT 2202)
13/06	1945	02:10	46.60	112.03	0.0	0	2	0. 1	HELENA (ALSO AT 1215)
15/06	1945	14:23	46.60	112.03	0.0	0	3	0. 1	HELENA
24/06	1945	03:35	46.60	112.03	0.0	0	2	0. 1	HELENA
26/06	1945	09:05	46.60	112.03	0.0	0	3	0. 1	HELENA (ALSO AT 1028)
04/07	1945	02:12	46.60	112.03	0.0	0	4	0. 1	HELENA
06/07	1945	16:01	46.60	112.03	0.0	0	4	0. 1	HELENA
14/07	1945	13:12	46.60	112.03	0.0	0	2	0. 1	HELENA
15/07	1945	14:19	46.60	112.03	0.0	0	2	0. 1	HELENA
26/07	1945	09:11	46.60	112.03	0.0	0	2	0. 1	HELENA
27/07	1945	11:50	46.60	112.03	0.0	0	2	0. 1	HELENA
27/07	1945	15:14	46.60	112.03	0.0	0	2	0. 1	HELENA
10/08	1945	09:57	46.60	112.03	0.0	0	2	0. 1	HELENA
22/08	1945	16:57	46.60	112.03	0.0	0	2	0. 1	HELENA
23/08	1945	20:35	46.60	112.03	0.0	0	2	0. 1	HELENA
25/08	1945	02:57	46.60	112.03	0.0	0	2	0. 1	HELENA
25/08	1945	04:50	46.60	112.03	0.0	0	2	0. 1	HELENA
28/09	1945	16:10	46.60	112.03	0.0	0	2	0. 1	HELENA
02/10	1945	11:10	46.60	112.03	0.0	0	3	0. 1	HELENA
28/11	1945	06:38	46.60	112.03	0.0	0	3	0. 1	HELENA
28/11	1945	06:44	46.60	112.03	0.0	0	2	0. 1	HELENA
06/12	1945	05:20	46.60	112.03	0.0	0	2	0. 1	HELENA
22/01	1946	06:42	46.60	112.03	0.0	0	2	0. 1	HELENA
08/03	1946	13:10	46.60	112.03	0.0	0	2	0. 1	HELENA
25/03	1946	08:25	46.60	112.03	0.0	0	1	0. 1	HELENA
07/06	1946	18:42	46.60	112.03	0.0	0	2	0. 1	HELENA
08/06	1946	04:25	46.60	112.03	0.0	0	2	0. 1	HELENA
12/06	1946	12:21	46.60	112.03	0.0	0	2	0. 1	HELENA
16/07	1946	08:00	46.60	112.03	0.0	0	2	0. 1	HELENA
03/08	1946	01:07	46.60	112.03	0.0	0	3	0. 1	HELENA
11/09	1946	01:42	46.60	112.03	0.0	0	3	0. 1	HELENA
11/09	1946	19:24	46.60	112.03	0.0	0	2	0. 1	HELENA
23/09	1946	22:02	46.60	112.03	0.0	0	2	0. 1	HELENA
29/09	1946	22:45	46.60	112.03	0.0	0	2	0. 1	HELENA
09/10	1946	14:00	46.60	112.03	0.0	0	2	0. 1	HELENA
10/10	1946	19:15	46.60	112.03	0.0	0	2	0. 1	HELENA
10/10	1946	19:50	46.60	112.03	0.0	0	2	0. 1	HELENA
02/11	1946	08:12	46.60	112.03	0.0	0	2	0. 1	HELENA
02/11	1946	17:22	46.60	112.03	0.0	0	2	0. 1	HELENA

Region 4—Helena-Ovando (continued)

D A T E	TIME	LAT	LONG	MAG	C	I N	AREA	S	C O M M E N T S
08/12 1946	07:10	46.60	112.03	0.0	0	2	0. 1	HELENA	
14/03 1947	11:00	47.20	113.50	0.0	0	6	0. 1	SEELEY LAKE	
15/03 1947	10:29	46.60	112.03	0.0	0	3	0. 1	HELENA	
30/05 1947	22:40	46.60	112.03	0.0	0	3	0. 1	HELENA	
05/07 1947	04:00	46.60	112.03	0.0	0	2	0. 1	HELENA	
22/07 1947	06:08	46.60	112.03	0.0	0	2	0. 1	HELENA	
10/08 1947	09:21	46.60	112.03	0.0	0	4	0. 1	HELENA	
03/10 1947	04:50	46.60	112.03	0.0	0	2	0. 1	HELENA	
12/12 1947	12:55	46.60	112.03	0.0	0	1	0. 1	HELENA	
15/12 1947	07:38	46.60	112.03	0.0	0	1	0. 1	HELENA	
17/12 1947	05:38	46.50*	112.00*	0.0	0	5	0. 1	OVANDO, LEWIS & CLARK COUNTY	
22/12 1947	13:15	46.60	112.03	0.0	0	2	0. 1	HELENA	
27/12 1947	01:48	46.60	112.03	0.0	0	2	0. 1	HELENA	
04/01 1948	05:30	46.60	112.03	0.0	0	3	0. 1	KENWOOD	
05/01 1948	04:26	46.60	112.03	0.0	0	4	0. 1	HELENA	
09/01 1948	11:27	46.60	112.03	0.0	0	2	0. 1	HELENA	
24/01 1948	14:50	46.60	112.03	0.0	0	2	0. 1	HELENA	
08/03 1948	03:40	46.60	112.03	0.0	0	3	0. 1	KENWOOD	
08/03 1948	07:30	46.60	112.03	0.0	0	2	0. 1	HELENA	
25/03 1948	03:55	46.60	112.03	0.0	0	2	0. 1	HELENA	
08/04 1948	02:30	47.07	113.12	0.0	0	5	0. 1	OVANDO (3 TREMORS)	
19/07 1948	15:18	46.60	112.03	0.0	0	3	0. 1	HELENA	
12/08 1948	21:29	46.60	112.03	0.0	0	3	0. 1	HELENA	
27/09 1948	14:51	46.60	112.03	0.0	0	3	0. 1	HELENA	
19/10 1948	12:57	46.60	112.03	0.0	0	5	0. 1	HELENA	
21/11 1948	15:41	46.60	112.03	0.0	0	4	0. 1	HELENA	
21/11 1948	15:43	46.60	112.03	0.0	0	4	0. 1	HELENA	
21/11 1948	16:03	46.60	112.03	0.0	0	2	0. 1	KENWOOD	
02/12 1948	15:15	46.60	112.03	0.0	0	2	0. 1	KENWOOD	
16/12 1948	02:15	46.60	112.03	0.0	0	3	0. 1	HELENA	
15/01 1949	05:21	46.60	112.03	0.0	0	2	0. 1	HELENA	
28/01 1949	20:27	46.60	112.03	0.0	0	4	0. 1	HELENA	
27/02 1949	11:41	46.60	112.03	0.0	0	2	0. 1	HELENA	
22/05 1949	14:23	46.60	112.03	0.0	0	2	0. 1	HELENA	
05/06 1949	05:49	46.77	111.90	0.0	0	4	0. 1	HAUSER DAM (15 MI N. OF HELENA)	
09/09 1949	21:42	46.60	112.03	0.0	0	3	0. 1	HELENA	
30/04 1950	09:48	46.60	112.03	0.0	0	3	0. 1	KENWOOD	
18/08 1950	00:22	46.60	112.03	0.0	0	4	0. 1	HELENA	
19/08 1950	18:44	47.25*	113.50*	0.0	0	6	10. 1	NEAR SEELEY LAKE	
15/01 1951	01:27	46.60	112.03	0.0	0	4	0. 1	HELENA	
28/01 1951	07:45	46.60	112.03	0.0	0	2	0. 1	HELENA	
29/01 1951	13:07	46.60	112.03	0.0	0	3	0. 1	HELENA	
23/04 1951	13:57	46.60	112.03	0.0	0	4	0. 1	HELENA	
20/05 1951	13:00	46.60	112.03	0.0	0	3	0. 1	HELENA	
14/11 1951	05:14	46.60	112.03	0.0	0	4	0. 1	HELENA	
09/12 1951	00:08	46.60	112.03	0.0	0	5	0. 1	HELENA	
11/12 1951	20:52	46.60	112.03	0.0	0	3	0. 1	HELENA	
03/01 1952	19:34	46.60	112.03	0.0	0	4	0. 1	HELENA	
12/02 1952	14:50	46.60	112.03	0.0	0	2	0. 1	HELENA	
26/02 1952	10:58	46.60	112.03	0.0	0	4	0. 1	HELENA	
26/04 1952	11:20	46.60	112.03	0.0	0	3	0. 1	HELENA	
10/07 1952	17:29	46.60	112.03	0.0	0	4	0. 1	HELENA	
19/07 1952	06:38	46.60	112.03	0.0	0	3	0. 1	KENWOOD	
02/09 1952	04:45	46.87	112.94	0.0	0	3	0. 1	HELMVILLE	
14/12 1952	00:58	46.60	112.03	0.0	0	2	0. 1	HELENA	
08/03 1953	18:39	46.60	112.03	0.0	0	4	0. 1	HELENA	
11/05 1953	09:56	46.60	112.03	0.0	0	3	0. 1	HELENA	
23/11 1953	06:30	46.87	112.94	0.0	0	5	0. 1	HELMVILLE	
03/07 1954	18:01	46.60	112.03	0.0	0	2	0. 1	HELENA	
23/09 1955	10:30	46.60	112.03	0.0	0	3	0. 1	HELENA	
24/09 1955	05:21	46.60	112.03	0.0	0	4	0. 1	HELENA	
05/11 1955	07:21	46.60	112.03	0.0	0	4	0. 1	HELENA	
02/02 1956	08:02	46.60	112.03	0.0	0	3	0. 1	HELENA	
03/03 1956	07:04	46.60	112.03	0.0	0	4	0. 1	HELENA	
28/10 1956	14:21	46.60	112.03	0.0	0	3	0. 1	KENWOOD	
29/10 1956	08:11	46.60	112.03	0.0	0	3	0. 1	KENWOOD	
18/11 1956	23:18	46.60	112.03	0.0	0	4	0. 1	HELENA	
25/11 1956	18:02	46.60	112.03	0.0	0	4	0. 1	KENWOOD	
25/12 1956	06:57	46.60*	112.03*	0.0	0	5	0. 1	HELENA	

Region 4—Helena-Ovando (continued)

D A T E	T I M E	L A T	L O N G	M A G	C	I N	A R E A	S	C O M M E N T S
11/11 1957	00:49	46.50*	112.00*	0.0	0	0	0.	0	
07/02 1958	20:37	46.60	112.03	0.0	0	4	0.	1	HELENA
11/05 1958	16:40	46.60	112.03	0.0	0	3	0.	1	HELENA (WEST SECTION)
28/05 1958	05:00	46.33	113.30	0.0	0	0	0.	1	PHILLIPSBURG
28/05 1958	09:45	46.50	113.00	0.0	0	6	12.	1	GEORGETOWN LAKE, PHILLIPSBURG
01/08 1958	08:27	46.33	113.30	0.0	0	4	0.	1	PHILLIPSBURG
13/10 1958	02:00	47.00*	112.50*	0.0	0	0	0.	0	
19/12 1958	08:30	46.60	112.03	0.0	0	4	0.	1	HELENA (ALSO AT 0835)
18/01 1959	20:41	46.60	112.03	0.0	0	3	0.	1	HELENA
19/04 1959	23:10	46.60	112.03	0.0	0	4	0.	1	HELENA
17/05 1959	03:56	47.50*	113.00*	0.0	0	5	8.	1	BOB MARSHALL WILDERNESS, NE OF SEELEY LAKE
14/01 1960	11:27	46.60	112.03	0.0	0	4	0.	1	HELENA
02/11 1961	22:50	46.60	112.03	0.0	0	4	0.	1	HELENA
01/02 1962	10:00	47.02	113.12	0.0	0	4	0.	1	ONE MILE S.W. OF OVANDO
01/06 1963	08:37	46.63	112.08	0.0	0	0	0.	1	FORT HARRISON
07/09 1963	16:30	46.65	112.10	0.0	0	4	0.	1	6 MILES N. OF HELENA, MARYSVILLE
01/01 1964	06:54	46.60	112.03	0.0	0	4	0.	1	HELENA
02/08 1964	10:30	46.60	112.03	0.0	0	2	0.	1	HELENA
13/08 1964	14:51	46.50*	112.20*	4.1	2	4	0.	1	HELENA
11/12 1964	17:22	46.70*	112.80*	0.0	0	0	0.	0	
06/01 1965	17:30	46.63	112.53	0.0	0	0	0.	1	WALKERVILLE (BUTTE SUBURB), (BETWEEN 1730 AND 1800)
13/04 1965	00:59	46.90*	113.10*	4.0	2	0	0.	0	
07/06 1965	10:59	46.90*	113.20*	0.0	0	0	0.	0	
26/10 1965	04:28	47.40*	113.20*	4.0	2	5	0.	1	SEELEY LAKE
04/03 1966	00:03	46.60	112.03	0.0	0	3	0.	1	HELENA
07/03 1966	11:09	46.30*	111.50*	4.8	2	5	13.	1	NEAR CLARKSTON
03/04 1966	09:22	46.60	112.03	0.0	0	4	0.	1	HELENA
03/10 1966	20:30	46.87	112.94	0.0	0	4	0.	1	HELMVILLE
22/01 1967	02:00	46.92	113.42	0.0	0	4	0.	1	GREENOUGH (ALSO AT 0300)
28/02 1967	12:52	46.61*	112.34*	0.0	0	3	0.	1	HELENA
19/04 1968	19:35	47.10*	113.00*	4.1	2	0	0.	0	
17/07 1968	11:26	46.77	112.27	0.0	0	4	0.	1	MARYSVILLE
20/11 1968	18:06	47.31*	112.76*	3.8	2	0	0.	0	
30/04 1969	20:10	46.70*	112.80*	3.9	2	5	8.	1	LINCOLN, ROGERS PASS
28/12 1969	22:20	47.32*	113.07*	4.3	2	0	0.	0	
30/03 1970	01:14	46.60	112.03	0.0	0	4	0.	1	HELENA (ALSO AT 1604)
26/07 1970	01:30	46.60	112.03	0.0	0	4	0.	1	HELENA (ALSO BUTTE)
14/11 1970	19:39	46.90*	113.34*	0.0	0	0	0.	0	
26/01 1971	03:52	46.43*	112.20*	0.0	0	4	0.	1	HELENA, 15 KM DEPTH
17/02 1971	00:48	46.60	112.03	0.0	0	4	0.	1	HELENA (ALSO BUTTE 0054)
22/06 1971	11:19	46.60	112.03	0.0	0	4	0.	1	HELENA (ALSO BUTTE)
05/01 1973	05:35	46.47*	112.73*	0.0	0	0	0.	0	
18/07 1975	08:06	46.72*	112.12*	3.9	4	4	0.	1	BIG ARM, FORT HARRISON, HELENA, MARYSVILLE (ALSO AT 1506)
18/07 1975	11:39	46.69*	112.13*	3.1	4	2	0.	1	HELENA AREA (ALSO AT 1839)
19/07 1975	05:00	46.69*	112.10*	3.5	4	2	0.	1	HELENA AREA (ALSO AT 1200)
11/12 1975	04:50	47.35*	113.15*	3.6	3	0	0.	0	
11/12 1975	09:01	47.39*	113.13*	3.8	3	0	0.	0	
12/02 1976	23:13	46.75*	112.13*	3.8	4	4	0.	1	CLANCY, EAST HELENA, MARYSVILLE, 1 KM DEPTH
04/09 1977	13:54	46.60*	112.14*	3.2	4	4	0.	1	HELENA; FELT AT CANYON FERRY, EAST HELENA
24/03 1978	11:35	47.21*	113.30*	3.8	4	0	0.	0	
23/04 1978	16:24	46.97*	113.27*	4.9	4	5	94.	1	OVANDO, ML=4.9
23/04 1978	16:36	47.00*	113.31*	3.7	4	2	0.	1	OVANDO
19/05 1978	03:50	47.00*	113.31*	3.2	4	0	0.	0	
13/09 1978	20:30	46.99*	113.31*	3.0	4	0	0.	0	
07/10 1978	05:35	46.62*	112.13*	2.9	3	4	0.	1	HELENA
15/10 1978	18:15	47.09*	113.16*	3.4	4	2	0.	1	OVANDO
21/10 1978	05:28	46.97*	113.25*	3.5	4	3	0.	1	OVANDO
10/11 1978	02:53	47.01*	113.33*	4.3	4	5	0.	1	OVANDO, FELT SEELEY LAKE
11/11 1978	17:29	46.50*	112.70*	3.4	4	0	0.	0	
04/01 1979	07:51	47.31*	113.14*	3.0	4	0	0.	1	N OF OVANDO
04/05 1979	11:58	47.09*	112.79*	3.5	4	0	0.	1	NEAR LINCOLN

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Joan Duncan, Helena City Commissioner

Bob Erickson, City Manager, Helena

Dick Nisbet, Director Public Works, Helena

Charlie Dickert, Water Superintendent, Helena

Glen McDonald, Montana Department of Natural Resources
and Conservation

Larry Cronenwett, U.S. Forest Service, Helena National Forest

Roger White, U.S. Forest Service, Northern Region

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